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THE SCIENCE

OF

BUILDING.

WORKS WRITTEN OR EDITED

BY

E. WYNDHAM TARN, M.A.

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THE SCIENCE

OF

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ON

THE PRINCIPLES OF CONSTRUCTION

Especially Adapted to the Requirements of Architectural Students.

By E. WYNDHAM TARN, M.A. (Lond.)

ARCHITECT.

Author of "Practical Geometry for the Architect, &c."

SECOND EDITION, REVISED AND ENLARGED.

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PREFACE

TO THE SECOND EDITION.

In the present Edition of this work the whole of it has undergone a thorough revision, and much of it has been re-written. A considerable amount of new matter has also been added, which it is hoped will increase its usefulness both to the Student and also to the practitioner of Architecture.

The following is an outline of the additions and alterations that have been made by the Author.

In Chapter I. the additional subjects are, *reaction* and the force of *gravity*, the practical application of the triangle of forces to show the principle of the *bracket*, the method of finding the centre of gravity of a section of an *iron girder* and of an *arc* or *sector* of a circle, the effect of *corbelling-out* and of *buttresses* on the stability of structures; the mode of investigating the *moment of resistance* in a beam is explained, and the nature of *Friction* and *Cohesion*.

Chapter II. on Retaining walls has been re-written, the method of obtaining the formulæ explained, and a

Table given of the requisite thickness of retaining walls for supporting different kinds of earth.

In Chapter III. has been added the *geometrical* method of determining the thrust of an arch, and the various formulæ for calculating the stability of arches, vaults and domes, have been much simplified; the subject of *conical domes* or *spires* has also been introduced and their stability determined, as well as the mode of constructing *iron domes.*

In Chapter IV. much new matter has been introduced on the *chemical composition and strength* of stones and the *geological formations* in which they are found; also on the subject of *mortar* and *cement.*

In Chapter V. is explained the mode of finding the *safe-load* on a beam with a given strain on the fibres, and the scantlings of floor-timbers have been calculated for various spans; the principles of the *hammer-beam* roof and of the *arched* roof have been investigated, and also the nature of the forces developed in *shoring-up* a wall.

Chapter VI. has been considerably enlarged by the addition of matter on the *nature* of iron, the method of finding the *safe-load* on a beam with a given strain on the fibres, the stability of iron roofs of a *circular* form, and the modes of *preserving* iron from decay.

Chapter VII. has been for the most part re-written, the subject of *Fluids at rest and in motion* being substituted for that of "Water in Vessels

and Pipes;" *fluids* including *air* as well as *water*; the nature and character of water is discussed, as well as its pressure upon the sides of reservoirs and tanks, and the method of conveying *hot-water* from the basement to the upper parts of a house; the quantity of water that can be discharged from the bottom of a tank with a given area of orifice, the resistance of long pipes and of *bends* or *elbows* therein to the flow of water, and the determination of the *strength of pipes* of different material. The nature and composition of *air* are described, as well as the law of *Diffusion* and the principles of *Ventilation*; and the effects produced on walls and roofs by the *pressure of wind*, moving with different rates of velocity, are discussed in detail.

Chapter VIII. is entirely new, and treats upon the subject of the nature of *lightning* and the methods to be adopted for *protecting* buildings from its violence.

The Appendix contains an additional Table of the *powers of numbers*, intended to facilitate the calculation of several of the formulæ given in the work.

PREFACE

TO THE FIRST EDITION.

In the following pages the Author has endeavoured to introduce the Student of Architecture to a general outline of the scientific subjects connected with his Profession, an acquaintance with which can at present be only obtained by the perusal of a large number of works by various authorities. In compiling this Treatise, the Author has been careful in all cases to consult the writers who stand highest in their respective branches of knowledge; and by avoiding the use of the higher mathematics, as well as those topics which belong more especially to the engineering profession, he has brought the various subjects within the capacity of those whose mathematical attainments do not extend beyond elementary geometry and algebra.

CONTENTS.

CHAPTER I.

MECHANICAL PRINCIPLES.

CHAPTER II.

RETAINING WALLS.

CHAPTER III.

ARCHES; CUPOLAS; SPIRES.

CHAPTER IV.

BUILDING STONES.

CHAPTER V.

TIMBER.

CHAPTER VI.

IRON.

CHAPTER VII.

FLUIDS AT REST AND IN MOTION.

CHAPTER VIII.

LIGHTNING CONDUCTORS.

APPENDIX.

ERRATA.

Page 40. Fifth column of Table, for "cos" *read* "cos. ϕ."
,, 190. In Table, against 30°, for "·125" *read* "·250."
,, 191. Line 7, for "one-sixth" *read* "one-third."
,, ,, Line 8, for "4lbs." *read* "8lbs."

APPENDIX.

THE

SCIENCE OF BUILDING.

CHAPTER I.

MECHANICAL PRINCIPLES.

1. The science of "mechanics" investigates the laws of forces and their effects upon various bodies. The term "force" is applied to whatever causes or *tends* to cause motion, or a change of motion in bodies, and to whatever prevents or hinders their motion; its effect being measured by reference to some unit of weight, as pounds, tons, &c. When a body is set in motion by a force, the effect of the force is measured by the momentum, or the mass of the body multiplied by the velocity generated in an unit of time. When two or more forces act upon a body, but do not produce motion, the body is said to be in *equilibrium* by the action of those forces. The relative magnitude of the forces acting on a body may be represented by straight lines whose lengths are proportional to the forces, and whose directions are those in which the forces act.

Whenever a force is impressed upon a body in any direction, another force of equal magnitude is immediately produced in the opposite direction, which is called the Reaction of the impressed force. Thus, if a heavy body

B

is placed on the ground, it presses thereon with a certain force W, which is proportional to the quantity of matter it contains, but the ground which supports it reacts upon the body with an equal pressure R acting upwards, otherwise the body would sink into the ground. If the body is placed on a liquid it will sink below the surface with a force equal to $W - R$, R in this case being only the weight of the volume of the liquid which the solid body displaces. Also when a body A, which is in motion, strikes a fixed body B, which brings it to a standstill, the stoppage of A is owing to a blow which it receives from B, equal in intensity to that which it struck B, otherwise it would have moved B along with it, or have passed through it as a bullet does through a plank of wood, where the resistance of the plank is less than the force of the blow. It is the reaction of the earth that gives stability to our buildings, and we also see the necessity of so distributing the load that this reaction shall be equal to the pressure, as a want of attention to this is the cause of numerous settlements in buildings from their foundations sinking into the ground.

One of the most important forces with which the architect has to deal is the *force of gravity*, which is measured by the velocity generated in one second by a body falling from rest in consequence of the earth's attraction, the amount of which is determined by experiment for every place on the globe. In this country it is found that a body falling from rest in a vacuum acquires a velocity in the first second of time of about 32·2 feet per second, for which quantity the letter g is always used, and the velocity v generated in any other time t is expressed by the formula

$$v = gt.$$

The space or height through which a body falls from

rest in one second is $\frac{1}{2}g$ or 16·1 feet, and the height h passed through by a body falling from rest during t seconds is (in feet)

$$h = \tfrac{1}{2} g t^2 = 16{\cdot}1\ t^2.$$

The velocity v which the body acquires in falling through the height h from rest, in feet per second, is

$$v = \sqrt{2gh} \;=\; 8\sqrt{h}, \text{ very nearly.}$$

It is customary to call h the height *due* to the velocity v.

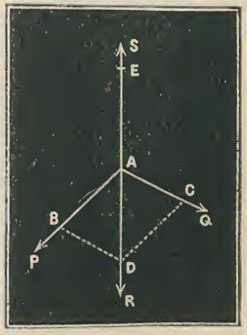
The quantity g is also put for the number of pounds weight in a *unit of mass*, so that if M represents the mass of a body whose weight in lbs. is W, we have

$$W = M.\,g, \text{ or } M = \frac{W}{g},$$

which latter quantity or ratio does not change with the latitude of the place, although the value of g may vary.

2. Resolution of Forces.—Whatever number of forces act upon a body, it is clear that they can only move it in *one* direction, so that their total effect must be equal to some one force acting in that *one* direction. This force is called the "resultant" of all the forces which act on the body, and if a force equal in magnitude to the resultant and opposite in direction is applied to the body it will be kept in equilibrium. Suppose the two forces P and Q (fig. 1) represented in magnitude and direction by the lines AB and AC, to act on a body A; draw CD parallel to AB, and BD parallel to AC; join AD; then the line AD represents

Fig. 1.



the resultant (R) of P and Q in magnitude and direction. Produce the line DA and take AE equal to AD, then AE will be the magnitude of the force (S) acting on A, which will just balance R, the resultant of P and Q.

The forces P and Q are called the "components" of the force R. If the single force R acts on the body A in the direction AD, and we want to know its effect in any two other given directions AB and AC, we take the length AD to represent R and make it the diagonal of a parallelogram whose sides are parallel to the given directions of AB and AC, then these sides will represent in magnitude the effect of R in those two directions.

If the angle BAC is a *right-angle*, then DB will be perpendicular to AB, and the ratio of DB to DA is called the *sine* of the angle DAB, that of BA to DA being called the *cosine* of DAB; the ratio of DB to BA is called the *tangent* of the angle DAB. The values of the sine, cosine, and tangent of every angle from 0° to 90° have been calculated, and will be found in tables. In this case the resolved parts P and Q, of the force R, at right angles to one another, are:—

$$P = R \times \text{cosine PAR} = R \times \cos. PAR$$
$$Q = R \times \text{sine PAR} = R \times \sin. PAR$$
$$Q = P \times \text{tangent PAR} = P \times \tan. PAR$$

When more than two forces act on the body at A, we can obtain the value and direction of their resultant by first finding that of any two of the given forces, and then considering those two as replaced by their resultant; then find the resultant of this last force combined with another of the given forces, and so on for all the forces, until at last we reduce them to a single force, which will be the resultant required. When the body A is kept in equili-

brium by the three forces P, Q, and S, represented by the lines AB, AC, AE; these forces are proportional each to the *sine* of the angle contained between the other two; that is to say:

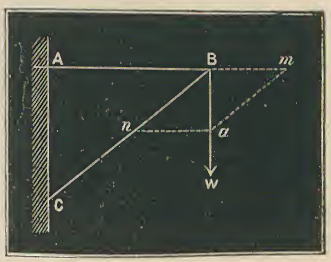
$$\frac{P}{\sin.\ QAS} = \frac{Q}{\sin.\ PAS} = \frac{S}{\sin.\ PAQ}.$$

An example of the practical application of this theorem will be found in the case of arches arranged in a polygon (20). Also if we form the triangle ABD of lines drawn in the directions of the forces, it is evident that these lines will represent the forces in magnitude and direction, if taken in order. That is to say, P is represented by AB, acting from A towards B; Q by BD, acting from B towards D; S by DA, acting from D towards A. And conversely, if three forces acting at a point are represented by the sides of a triangle, taken in order, they will balance each other.

It is also clear that in order that three forces may balance each other or be in equilibrium, their directions must meet in one point.

The following is an example of the application of the principle of the Resolution of forces to practical building.

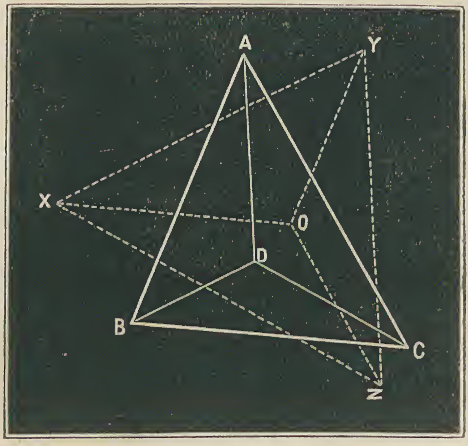
Fig. 2.



Let AC (fig. 2) represent the vertical face of a wall, in which a beam AB is fixed firmly at A, but projects therefrom the length AB, the end B having to sustain a weight W, which would bend it downwards, but is prevented from doing so

by the strut BC fixed to the end B of AB, and into the wall at C, so as to form what is called by architects a "bracket." Let the weight W be measured by the length of the vertical line Ba on any convenient scale of parts. Draw am parallel to BC, meeting AB produced in m; and draw an parallel to Bm, and meeting BC in n. The length Bm will then represent the force with which AB is strained in the direction Bm, on the same scale that Ba represents the load W. Also the length Bn will represent on the same scale the pressure down the strut in the direction BC; the former being a stretching or tensile force, and the latter a compressive one.

Fig. 3.

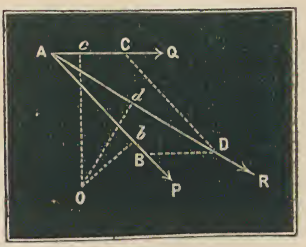


3. Let ABC (fig. 3) be a triangular piece of framework jointed at A, B, and C, and kept in equilibrium by the action of three forces acting at the joints. Since there is equilibrium the directions of these forces must meet at some point, D, and must also be proportional to the sides of a

triangle which are respectively parallel to the lines AD, BD, CD. Let XYZ be such a triangle. Then to find the stresses in AB, BC, CD arising from the forces, draw lines from the points X, Y, Z, parallel respectively to the sides of the triangle ABC; these will meet in a point O, and the line XO will represent the stress in BC; YO that in AB; ZO that in AC; to the same scale as that on which the sides of the triangle XYZ, represent the original forces acting at the joints A, B, C. The figure ABC is called the "frame diagram," and the figure XYZ the "stress diagram." The lines which in the former make a closed polygon are represented in the latter by lines meeting in a point, and vice versâ; so that they may be called *reciprocal figures*. This is the basis of the method of diagrams applied by the late Prof. Clerk Maxwell to the determination of the relative values of the stresses on the several parts of any framework, such as a roof-truss, lattice-girder, &c. (See paper by J. C. Maxwell read at meeting of British Association, 1867.) An example of its application to iron roofs will be found at section (**67**).

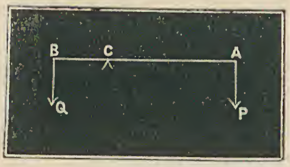
4. Moments.—It is often necessary in the solution of mechanical problems to ascertain the effect of a force upon a body with respect to some point through which its direction does *not* pass. Let the forces P, Q, and R act at A (fig. 4), and be represented by the lines AB, AC, AD. Let O be some point which is not in the direction of either of these forces; draw O*b* perpendicular to AB; O*c* to AC; O*d* to AD; then the product of the force P into the length O*b* is called the "moment of P about O;" the

Fig. 4.



product of Q into Oc is the *moment of Q about O*; and the product of R into Od is the *moment of R about O*. Moreover, by the principles of geometry it can be proved that the moment of the resultant R about O is equal to the *sum* of the moments of P about O and of Q about O. The *moment* of a force about any point measures its tendency to produce rotation round that point. Whenever two forces act in any directions upon a body having a fixed point, they will be in equilibrium if their moments about the fixed point are equal, but tending to rotation in opposite directions. Thus, if we have a straight rod AB (fig. 5) resting on a fixed point C, and a weight P is applied at one end A, and a weight Q at the other end B, these weights will balance each other if P $\times$ AC = Q $\times$ BC.

Fig. 5.



In this we have the principle of the lever, for by moving C, the fulcrum, near to B, we can make a small weight at A balance a large one at B. The weight P which will balance a given weight Q is determined by the equation

$$P = \frac{BC}{AC} \times Q.$$

If the fulcrum C is placed at one end of the lever, and Q acts at a point B between A and C, the forces P and Q must be in opposite directions in order that they may balance, but the same relation exists between them; that is to say, P $\times$ AC = Q $\times$ BC.

The theory of moments is constantly applied in all the investigations into the principles of scientific construction, and therefore requires to be thoroughly understood by the architect.

5. Couples.—When two parallel forces P and Q act on a body in the same direction but at different points A and B (fig. 6), their resultant R is equal to their sum,

Fig. 6.

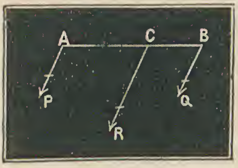
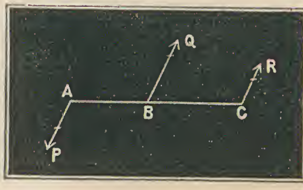


Fig. 7.



and acts at a point C which is determined by the equation,

$$P \times AC = Q \times BC.$$

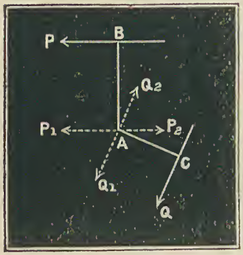
If the forces P and Q act in opposite directions (fig. 7) (Q being greater than P) the resultant R is equal to their difference and acts in the direction of the greater force at a point C, determined as before by the equation,

$$P \times AC = Q \times BC = Q\,(AC - AB),$$
$$\text{or } AC = \frac{Q}{Q - P} \times AB.$$

From this it will be seen that as P and Q approach equality, the point C goes further and further away, so that when P=Q, C is at an infinite distance, or there is no point of application of the resultant. The two equal and parallel forces acting at A and B in the same plane, now constitute what is termed a "couple;" and the moment of P about B, or of Q about A, is called the *moment of the couple*. The effect of a couple is to communicate an angular motion about an axis perpendicular to the plane in which its forces act, which axis passes through the *centre of gravity* (6) of the body. Two

couples which act upon a body in the same plane, but in opposite directions, will be in equilibrium if their moments are equal. The effect of a couple upon a body is not altered by increasing or diminishing the forces of the couple, provided the length of its arm is diminished or increased in like proportion, so that the moment or product of the force into the length of the arm, remains the same. The principle of couples enables us to shift the point of application of any forces acting on a body to a point about which they balance, an operation which is often necessary in mechanical problems, and enables us to simplify their solution. Suppose P and Q (fig. 8) to be two forces acting in the same plane on a body, and that their moments balance about a point A, so that

Fig. 8.



$$P \times AB = Q \times AC,$$

AB being perpendicular to the direction of P, and AC to that of Q. If we apply at A two equal and opposite forces, each equal and parallel to P, and call them P_1 and P_2, we shall not disturb the equilibrium; also if we apply at A two equal and opposite forces, Q_1 and Q_2, each equal and parallel to Q, the equilibrium is undisturbed. We now have the couples P. BA. P_2 and Q. CA. Q_2, acting in opposite directions, and having equal moments, so that they balance each other, and can be removed without affecting the equilibrium. We have, therefore, remaining only the forces P_1 equal to P, and acting parallel to its direction; and Q_1 equal to Q, also acting parallel to its direction; and these both act at the point

A. Hence it follows that if two forces, acting in any direction on a body, balance each other about any point in that body, they may be transferred to that point where they may be considered as acting with the same force as before, and in directions parallel to their original ones; and their effect upon the body is the same as before. This is an important principle in the investigation of the effect which the thrust of an arch produces on its supporting pier (18).

6. Centre of Gravity.—When two parallel forces act upon a body, we have seen in the last article that the point at which their resultant acts is found by equating their moments about that point. In the same way the position of the resultant of any number of parallel forces can be determined, by first finding the resultant of any two of the forces, and then of that first resultant and a third force; and so on until there is only one force left, which is the resultant of all the forces. Now every solid body which is subject to the action of the earth's attraction or gravitation may be considered as consisting of a number of heavy particles, the weights of which are parallel forces acting vertically downwards. There is therefore a point in every body about which all these parallel forces balance, and through which their resultant passes; this point is called the "centre of gravity" of the body. This statement is only strictly true for bodies which are very small as compared with the diameter of the earth, for the direction in which gravity or the earth's attraction acts is towards the *centre* of the earth, and the force of gravity also varies according to the latitude of the place; but as we have only comparatively small objects to deal with, we can, without any perceptible error, consider that the action of gravity on any solid

body is perpendicular to the surface of still water at that place, and is also uniform in force throughout the body. The position of the *centre of gravity* of any body is the same whatever may be the intensity of the force of gravity, and does not vary with change of position. In the case of a rigid body, we may, in all mechanical investigations, consider that its whole weight is collected at its centre of gravity, and if this point is supported, the whole body is in equilibrium.

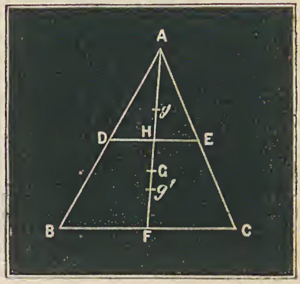
If a body is hung up loosely by any point, so that it can turn freely about that point, the centre of gravity will always be in a vertical or plumb-line dropt from the point of suspension; hence we can readily find the position of the centre of gravity in an irregular plane figure by suspending it at two points, and the intersection of the plumb-lines dropt from each point will give us the position of the centre of gravity.

In all regular figures, whose material is homogeneous, the centre of gravity is in the centre of the figure; as in the circle, ellipse, polygon of equal sides, sphere, ellipsoid, and all regular solids. In the square and parallelogram it is at the intersection of the diagonals drawn from opposite corners. In any triangle its position is found by bisecting one side and joining the point of bisection with the opposite vertex, and measuring one-third of this last line from the bisected side, or two-thirds from the vertex. Thus in the triangle ABC (fig. 9) AF bisects BC, and $GF = \frac{1}{3}$ AF, G being the centre of gravity of the triangle.

Knowing the position of the centre of gravity (G) of the triangle ABC we can find that (g') of a part BCED, cut off by the line DE parallel to BC. The centre of gravity of the smaller triangle ADE is g on the line AF,

and is found as above. Let g' be the centre of gravity of the part BCED, this will also be on the line AF (the point F bisecting BC). We may now consider the weight w of the triangle ADE as acting at g, and the weight w' of the figure BDEC as acting at g'; then the moments of their weights must balance about G, so that

Fig. 9.



$$w \times g\mathrm{G} = w' \times \mathrm{G}g', \text{ or } \mathrm{G}g' = \frac{w}{w'}\,\mathrm{G}g.$$

Now since w and w' are proportional to the areas of the figures, we may put the ratio of the areas for that of the weights. If A is the area of the whole triangle, a that of the smaller one, we have,

$$\mathrm{G}g' = \frac{a}{\mathrm{A} - a} = \mathrm{G}g.$$

Example.—Let $\mathrm{AE} = \frac{1}{2}\,\mathrm{AC}$; then $\mathrm{A} = 4a$

$$\mathrm{G}g = \tfrac{1}{3}\,\mathrm{AF}, \therefore \mathrm{G}g' = \tfrac{1}{3}\,\mathrm{G}g = \tfrac{1}{9}\,\mathrm{AF}$$
$$\mathrm{F}g' = \mathrm{FG} - \mathrm{G}g' = \tfrac{1}{3}\,\mathrm{AF} - \tfrac{1}{9}\,\mathrm{AF} = \tfrac{2}{9}\,\mathrm{AF}.$$

Another method of finding the centre of gravity of a trapezium, is to divide it into two triangles, then join their centres of gravity, and the distance of the centre of gravity of the figure from that of any one of the triangles is inversely proportional to the areas of the triangles. Thus if g is the centre of gravity of one triangle whose area is A (fig. 10), g' that of the other whose area is a, G the centre of gravity of the trapezium and situate

in the line gg', its position is determined by the equation,

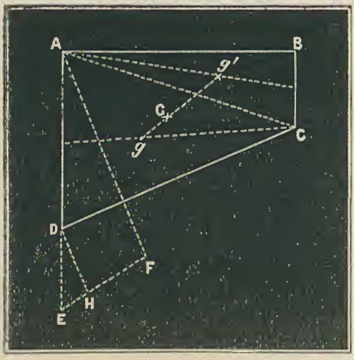
$$A \times Gg = a \times Gg', \text{ or } Gg' = \frac{A}{a} Gg = \frac{A}{a}(gg' - Gg')$$

$$\text{or } Gg' = \frac{A}{A + a} \times gg',$$

so that we have to divide the line gg' in the proportion of the two areas A and a.

Example.—Let ABCD (fig. 10) represent a trapezium having the two sides AD and BC perpendicular to AB; join AC; then the areas A and a are as the bases AD and BC; let g' be the centre of gravity of the triangle, ABC, and g that of ACD; join gg'. Produce AD to E, making DE equal to BC; at E draw a line EF, making any angle with AE, and take EF $= gg'$; join AF, and draw DH parallel to AF; then EF is divided in the ratio of A : a; and G is found by making gG = EH.

Fig. 10.



The general formula for the centre of gravity of a body which can be divided into two parts whose masses are M_1, and M_2, is

$$x = \frac{M_1 x_1 + M_2 x_2}{M_1 + M_2}$$

where x, x_1 and x_2 are the distances of the centres of gravity of the whole and of the two parts respectively,

measured from any fixed point that may be most convenient. Or if x_1 is required,

$$x_1 = \frac{(M_1 + M_2)\,x - M_2\,x_2}{M_1}.$$

Example.—Apply this formula to find g' in fig. 9, the point A being that from which x is measured; then since the masses of the triangles are as the squares on AF and AH,

$$Ag' = x_1 = \tfrac{2}{3}\,\frac{AF^3 - AH^3}{AF^2 - AH^2},$$

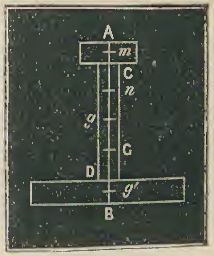
and if $AH = \frac{1}{2}\,AF$,

$$Ag' = \tfrac{2}{3}\,\frac{1 - \frac{1}{8}}{1 - \frac{1}{4}}\,AF = \tfrac{7}{9}\,AF$$

$$\therefore\ Fg' = AF - Ag' = \tfrac{2}{9}\,AF.$$

To find the centre of gravity (G) of any solid pyramid or cone whose vertex is A. First find g the centre of gravity of the base; join Ag, and take $gG = \frac{1}{4}\,Ag$. That of a thin hollow cone is found by taking $gG = \frac{1}{3}\,Ag$. That of a solid hemisphere is found by measuring $\frac{3}{8}$ of the radius above the centre. That of a thin hollow hemisphere is half way between the centre and the vertex.

Fig. 11.



Suppose it is required to find the centre of gravity G of the section of an iron girder (fig. 11), consisting of three rectangles whose respective areas are a, b, c, and of which m, g, and g' are the centres of gravity, and AB the vertical axis. Let n be the common centre of gravity of m and g, found from the equation

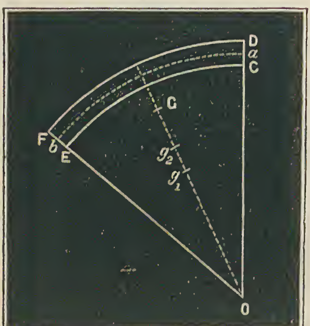
$$b.gn = a \times mn$$
$$= a\,(mg - gn)$$
$$\text{or, } gn = \frac{a}{a+b}\,mg,$$

which gives the position of the centre of gravity n of the two rectangles ACD. We can now find G the common centre of gravity of n and g', by the equation

$$c \times \mathrm{G}g' = (a+b)\,\mathrm{G}n$$
$$= (a+b)\,(ng' - \mathrm{G}g')$$
$$\text{or, } \mathrm{G}g' = \frac{a+b}{a+b+c}\,ng'$$
$$ng' = gn + gg'$$
$$\mathrm{BG} = \mathrm{B}g' + \mathrm{G}g'.$$

As an example, let $a = 3 \times 1$, $b = 6 \times 1$, $c = 8 \times 1\frac{1}{2}$; then $mg = 3\frac{1}{2}$, $ng = \frac{7}{6}$, $ng' = \frac{50}{12}$, $\mathrm{G}g' = \frac{50}{28}$, $\mathrm{BG} = \frac{20}{7}$

Fig. 12.



Let it be required to find the centre of gravity G of the circular arc EC (fig. 12), whose centre is O and radius r, subtending the angle EOC at O. The point G will be in the radius which bisects the angle EOC, and its distance from O varies as the *sine* of half the angle, and inversely as the angle. Putting K for this ratio we find G from the equation

$$\mathrm{OG} = \mathrm{K}\,.\,r,$$

the value of K for different angles being given in the following table :—

Angle EOC.	Value of K.
10°	·999
20	·995
30	·989
40	·980
50	·969
60	·955
70	·939
80	·921
90	·900

To find the position of g_1, the centre of gravity of the sector EOC, we have

$$Og_1 = \tfrac{2}{3} K . r.$$

If g_2 is the centre of gravity of the sector FOD, whose radius is R, then

$$Og_2 = \tfrac{2}{3} KR.$$

Having determined the centres of gravity of the two sectors we can, by means of our previous methods, find the centre of gravity G of the arch DCEF. For since the areas of the sectors are proportional to the squares of their radii, we have

$$Gg_2 = \frac{r^2}{R^2 - r^2} \times g_1 g_2$$

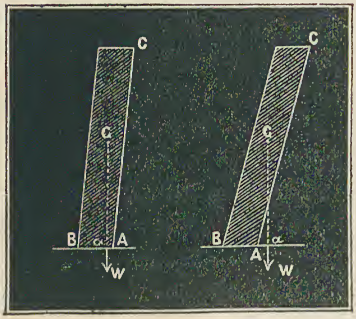
$$\text{where } g_1 g_2 = Og_2 - Og_1$$

$$\text{and } OG = Og_2 + Gg_2.$$

If the thickness DC of the arch is small, as compared with the radius OC, the position of G will very nearly correspond with the centre of gravity of an arc whose radius is $r + \frac{1}{2}$ DC, as shown by the dotted line *ab*.

7. Stability of Structures.—A knowledge of the position of the centre of gravity of any wall or other structure, enables us to determine whether it is in a condition of stability or of instability; that is to say, whether it is in such a condition as to be able to resist an additional force being applied to it in any direction without being overthrown; as well as the exact amount of force which will be required to overturn it. If a plumb-line dropt from the centre of gravity of a body falls *within* the base, that body is in a condition of stability, but if any force is applied to it which causes the plumb-line to fall *outside* the base, the body will be overturned. Let figs. 13 and 14 represent the section of a wall which is out of the perpendicular; the weight W acting at the centre of gravity will have a moment about A, namely $W \times Aa$, which in fig. 13 tends to keep it from falling, and in fig. 14 tends to overturn it.

Fig. 13. Fig. 14.



If a force F, equally distributed over its whole surface, such, for example, as that of the wind, is brought to bear on the wall in fig. 13, it will have a moment, $F \times \frac{1}{2} AC$, tending to overturn the wall about A; and if the value of this moment is greater than that of the moment $W \times Aa$, the wall will fall over. The pressure of the wind on a vertical surface is sometimes as great as 40 lbs. per square foot (**84**), and its resultant may be taken as acting at the middle of the wall. If, then, h is the height of the wall, the pressure of the wind (F) at

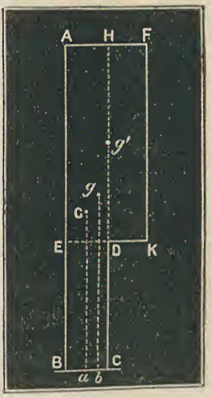
the middle for one foot length of wall is $40h$; and the moment of F about A is $40\frac{h^2}{2}$, or $20h^2$.

Let w be the weight per cubic foot of the wall, t its thickness, $Aa = x$; then the moment of the weight about A is $w.h.t.x.$; if this is greater than $20h^2$ the wall will stand, and if less it will fall over. For example, let $h = 40$ ft., $t = 5$ ft., $w = 110$ lbs. When the two forces just balance, we have

$$20 \times 1600 = 110 \times 40 \times 5 \times x; \text{ or } x = 1\tfrac{1}{2} \text{ ft.}$$

Therefore, in this case the plumb-line from G must fall more than $1\frac{1}{2}$ ft. within the base, otherwise the wall will be blown down by a storm of wind exerting a force of 40 lbs. per square foot of surface; so that the top of the wall must not in this case project more than 2 ft. beyond the base A. The stability of a lofty factory chimney may be determined in this manner. It can be shown (**84**) that the pressure of the wind on a circular chimney stalk is to that on a square one of the same diameter in the ratio of 2 to 3.

Fig. 15.



We will now apply the foregoing principles to a case which frequently occurs in building, namely, that of a mass of masonry "corbelled-out" from the face of a wall, as HK from the face HDC of the wall AC (fig. 15). Suppose the figure to represent the section of such a structure, the centre of gravity of the wall ABCH will be at G, and a vertical therefrom Ga will cut the base BC at

the point a, the wall being in a condition of stability. If we now add on the projecting portion HK, the centre of gravity of the whole mass will be moved to g, and the perpendicular gb will cut the base of the wall at b, which is nearer to the edge C than the point a, consequently the structure will not be in so stable a condition as it was without the projection, since the lever arm bC is less than the lever arm aC.

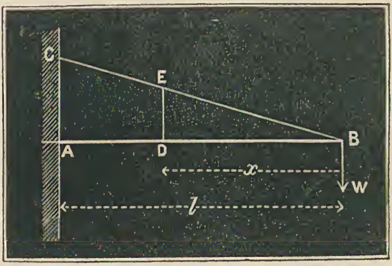
The "Metropolitan Building Act" allows a builder to corbel out a chimney stack above a certain height the full thickness of the wall, so that if HF = AH, the figure will represent the section of such a structure; and if we consider it with regard to the base line BC, it will have a moderate amount of stability. We will, however, suppose that there is a joint in the masonry at ED, and consider the mass above EK as simply resting on ED. The centre of gravity of this mass will be at g' on the line HD, and the weight will consequently just pass down the face of the wall, so that the least pressure against the back AE of the wall would overturn it about D, unless the cohesion of the bed of mortar or of cement at ED was sufficient to resist that pressure. We see then that a structure of this kind is in a condition bordering on instability, and is only kept from falling by the cohesion of the cement or by the resistance offered by the lateral extension of the wall on each side of the projection. Where the width of the projection is only small as compared with that of the wall, such a structure will stand in perfect safety; but where the chimney breast extends nearly the whole length of the wall, as is the case in some lofty London houses, there is a great tendency to overturn the wall about the line ED, which is only checked by the resistance offered by the other walls abutting at right angles to it; and if these abutments should at any

time be removed, there will be nothing to prevent the chimney-stack from pulling the wall over. It therefore is essential to the stability of walls that a limit should be put to the corbelling out of such projecting masses.

The effect produced by a "buttress" is just the opposite of what has been described above, and will be seen by reversing the figure ABCDK (fig. 15), and supposing AF to be the base and BC the top of the wall. In this case the moment of the weight of the wall and buttress DF about F increases in proportion to the width of base AF, while the moment of a horizontal force acting at the top B of the wall, and taken about F, remains the same; consequently the stability of the structure increases with the width of the base. In cases where the pressures come upon the wall at intervals instead of being uniformly distributed along the entire length, it is therefore more economical to throw out buttresses at the points of greatest pressure, than to increase the thickness of the whole wall.

8. Strains in Beams.—Let AB (fig. 16) be a beam of

Fig. 16.



length (l) fixed in a wall at A, and loaded with a weight (W) at B. Then the "strain" upon any part D of the beam at distance x from B is by (4) represented by the

moment of the weight (W) about the point D; that is, if we call M the moment of strain at D, we have

$$M = W . x$$

and the moment of strain at the point A, next the wall, is

$$M = W . l.$$

The strain, therefore, at any point is proportional to its distance from B. If we take CA vertical and make it represent the value of W. l on any convenient scale, and join CB, then the ordinate DE at any point (D) will represent the moment of strain (W . x) at that point to the same scale as AC represents W . l.

Suppose the weight (W) instead of being applied at one end of the beam AB, to be distributed uniformly over its entire length, w being the weight per unit of length, so that $w . l = W$. Then the weights acting on each unit of length will be so many equal and parallel forces, and their resultant (5), which is equal to their sum, must act at the middle of the beam. Now W equals the sum of all these forces, and its point of application is therefore at the centre of the beam. Therefore the moment of strain upon the point A is

$$M = W \times \frac{l}{2} = \frac{w}{2} l^2.$$

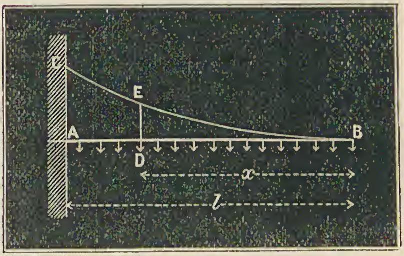
The moment of strain upon any other point D of the beam at distance x from B is

$$M = \frac{w}{2} x^2.$$

If, then, we draw AC (fig. 17) vertical and make it represent on any scale the value $\frac{1}{2}$ W . l, and draw a parabola CB having its vertex at B, the ordinate DE at

distance x from B will represent to the same scale the value of the moment of strain at the point D. If AC does not exceed $\frac{1}{4}$ AB, an arc of a circle may be substituted for the parabola BC, without material error.

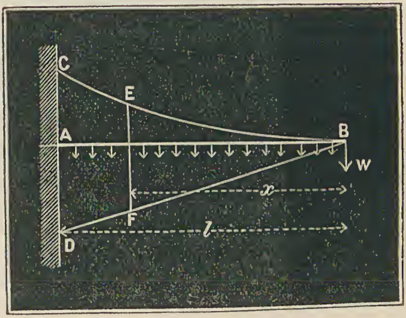
Fig. 17.



Comparing the strain produced by the weight W concentrated at B, with that produced by the same weight uniformly distributed, we see that the strain in the former case on any point is double the strain in the latter case, so that a beam will bear twice as much when the load is uniformly distributed as it will when the load is concentrated at one end.

When the beam AB is loaded at one end with a weight W, and has also a distributed load $w \cdot l$ to bear, the diagram representing the moment of strain at any point is a combination of the two previous figures. In fig. 18 let DAC be drawn at right angles to AB, take AD equal to W . l, AC equal to $\frac{1}{2}$ $w \cdot l^2$. Draw the parabola CB, or an arc of a circle if AC is less than $\frac{1}{4}$ AB, and join DB. Then the moment of strain at any point at distance x from B is represented by the

Fig. 18.



ordinate EF. The moment of the strain at that point is

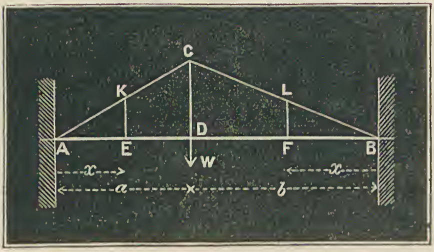
$$M = W \cdot x + \tfrac{1}{2} w \cdot x^2.$$

And the moment of the strain at A is

$$M = W \cdot l + \tfrac{1}{2} w \cdot l^2.$$

Now take the case of a beam AB (fig. 19) supported at the two ends and loaded at any intermediate point D at distance a from A, and distance b from B. Suppose the

Fig. 19.



forces P and Q represent the vertical reactions at A and B caused by the weight W. Then if the point where W acts is supposed to be a fixed fulcrum, and weights equal to P and Q are placed at A and B, the beam will be strained in the same manner as at first supposed, and $Q \cdot b$ will be the moment of strain at D. Also by taking moments about A we have

$$Q \cdot l = W \cdot a, \text{ or } Q = W \frac{a}{l}.$$

Therefore the moment of strain at D $= Q \cdot b = W \times \dfrac{a \cdot b}{l}$.

If x is the distance from A or B of the points E or F, the moment of strain at the point E is

$$M = W \frac{x \cdot b}{l},$$

and the moment of strain at the point F is

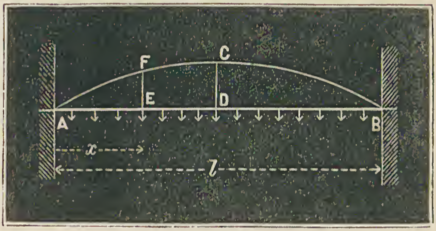
$$M = W \frac{x \cdot a}{l}.$$

To draw the diagram of strains, take DC vertical and equal on scale to $W \frac{ab}{l}$; join CA, CB, then the ordinates EK, FL represent the moments of the strains at E and F.

If the point D is the middle of the beam, the moment of strain at that point is $\frac{1}{4} W \cdot l$; since $a = b = \frac{1}{2} l$, and the moment of strain at E or F is $\frac{1}{2} W \cdot x$.

Let the weight (W) be uniformly distributed over the whole length of the beam AB (fig. 20). Then it can be

Fig. 20.



shown by reasoning similar to that just employed, that the moment of strain at any point E at distance x from A or B is

$$M = \frac{W}{2} \times \frac{x(l - x)}{l} = \frac{1}{2} w \cdot x (l - x)$$

if $W = w \cdot l$, or w is the weight per unit of length; the

moment of strain at D is found by making $x = \frac{1}{2} l$, and is

$$M = \frac{1}{8} W . l = \frac{1}{8} w . l^2.$$

Comparing this with the last example, we see that the strain at the centre, when W is concentrated there, is twice as great as when it is uniformly distributed.

To draw the diagram for the strains produced by a distributed weight at any part of the beam, take DC to represent on scale the quantity $\frac{1}{8}$ W . l, draw a parabola ACB having its vertex at C; or if DC is not more than $\frac{1}{8}$ AB, a circular arc with its centre on the line CD produced may be substituted without material error. Then the ordinate EF will represent the moment of strain at any point E, to the same scale as DC is drawn.

For methods of drawing the parabola and other curves the reader is referred to the Author's Treatise on "Practical Geometry for the Architect."*

The delineation of diagrams in several other arrangements of load will be found in Humber's "Handy-book of Strains,"* but the above-mentioned are the principal ones that occur in ordinary buildings.

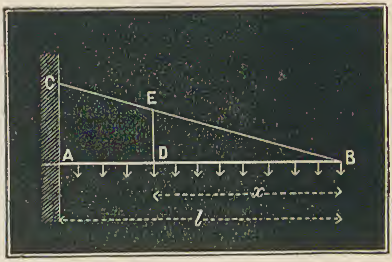
9. Shearing.—There is another kind of strain which is produced in a loaded beam, called the "shearing force," or the *vertical* action of the straining weight transmitted along the whole length of the beam, and tending to cause contiguous portions or sections to slide vertically upon each other.

When a beam is fixed at one end and loaded at the other with a weight W, that weight represents the *shearing force* produced at every point along the beam.

* Lockwood and Co.

If the weight ($W = w\,l$) is uniformly distributed along the beam AB (fig. 21), so that w is the weight on each unit of length; then the *shearing force* at any point D at distance x from B is $w \cdot x$, which is the load on the part of the beam between B and D. And the *shearing force* at A is W. If we take the vertical line AC to represent W, and join CB, then the ordinate DE will represent the *shearing force* at any point D.

Fig. 21.



When a beam AB of length l is supported at each end, and loaded with a weight W at any point C, at distances a and b from A and B, the *shearing force* developed at every point between C and A will be represented by $W\dfrac{b}{l}$; and at every point between C and B by $W\dfrac{a}{l}$. If C is the middle of the beam, then $a = b = \frac{1}{2}\,l$, and the *shearing force* at every point will be $\frac{1}{2}$ W.

Fig. 22.

When a beam AB (fig. 22) of length l is uniformly loaded with a weight ($W = w \cdot l$), the *shearing force* at A and B will be represented by $\frac{1}{2}\,w \cdot l$, and at any

point at distance x from either end it will be represented by $w\left(\frac{1}{2}\, l - x\right)$. Therefore, at the centre, where $x = \frac{1}{2}\, l$, the *shearing force* will be zero. If, then, the verticals AC and BD are taken to represent the quantity $\frac{1}{2}$ W, and we join CE and DE, the ordinate FK or GL will represent the *shearing force* at F or G.

10. NEUTRAL AXIS.—Whenever a beam of any elastic material is supported at its two ends in a horizontal position, and loaded in the middle, it is subjected to a transverse strain, whereby the fibres or particles in the upper part are compressed, whilst those in the lower part are extended. Also the fibres nearest the top are more compressed than those lower down in the beam; and the fibres near the bottom are, on the other hand, more extended than those which are higher up. Hence it follows that there must be a layer of fibres or particles in the interior of the beam, which is neither extended nor compressed; and this layer is called the "neutral surface" of the beam. If we take a vertical section through the middle of the beam, the line in which the neutral surface cuts that section is called the "neutral axis," and may be considered as passing through the centre of gravity of the section, provided the strain upon the beam is not sufficient to injure its elasticity. Some very carefully-made experiments by Mr. Barlow, which are described in the Transactions of the Royal Society and in Barlow's treatise on Strength of Materials,* show that in cast-iron beams of rectangular section the neutral axis is at the centre of the section of the beam even when strained with weights amounting to three-fourths of the breaking weight, and that its position is not sen-

* Lockwood and Co.

sibly altered by variations in the amount of the strain applied.

11. Elasticity.—When any substance is strained by a force applied in any manner, it will be found that its dimensions are altered during the application of the force; and such a body is said to be *perfectly elastic* if, when the action of the force ceases, it will return exactly to its original dimensions. No substance, however, can be considered perfectly elastic, since some permanent alteration in size and shape is produced whenever a strain is applied. Nevertheless, in practice the alteration is generally of so slight a character when the strain is much less than sufficient to produce fracture, that bodies are generally considered elastic so long as the strains to which they are subjected do not exceed a certain amount, called the "limit of elasticity." If we take a prism of any material having one square inch of section, and subject it to a longitudinal strain by a force S, the change in its length will be proportional to the amount of force applied, as long as the *limit of elasticity* is not exceeded. Let l represent the original length, x the change in length produced by the force S, E a constant depending on the nature of the material of which the prism is composed; then it is found that,

$$x : l :: S : E$$

$$\text{or,} \quad E = S \times \frac{l}{x}.$$

The constant E is called the "modulus of elasticity," and its value is determined by experiment for various substances. If we put $x = l$, we get $E = S$, or E is the force that would stretch a body, whose section is one square

inch, to double its length, if such a thing were possible.

12. Moment of Inertia.—If we suppose any area or plane surface to be divided into a very large number of indefinitely small particles, and call a the area of any one of such particles, x its distance from any given line or axis, then the product ax^2 is called the "moment of inertia" of that particle about the given axis, and the *moment of inertia* of the whole surface is the *sum* of all possible values of $a.x^2$. This is found by means of the integral calculus when the outline of the surface is a known geometrical figure. The formula for the *moment of inertia* of various geometrical figures is given in treatises on dynamics with respect to various axes. In the present work we shall only require to know it for two simple figures, and the axis will be taken through the centre of gravity.

Let the area of the figure be rectangular, having the depth d and breadth b; then the *moment of inertia* (I) of a rectangle about its centre of gravity is found to be

$$\text{I} = \tfrac{1}{12}\, bd^3.$$

If we have a hollow rectangular figure or a beam of I section (figs. 49, 50, 51)* in which t is the total thickness of the web or vertical portion, d_1 the clear depth between the top and bottom flanges, we find the moment of inertia about the centre of gravity to be,

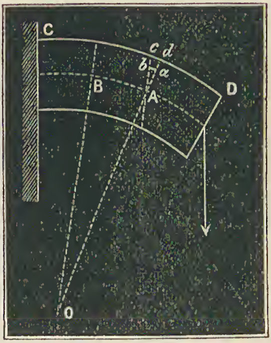
$$\text{I} = \tfrac{1}{12}\,(b\,.\,d^3 - (b-t)\,d_1^3).$$

When a beam is fixed at one end and strained by a load at the other end, it assumes a curved form as CD (fig. 23). Let AB be the neutral axis of the section (**10**), OA and OB the radius of curvature of AB, then the fibres above

* Page 145.

AB are extended and those below it are compressed in proportion to their distance from the neutral axis. Draw Ac parallel to OB. Let a be the area of section of any

Fig. 23.



fibre at a distance Ab = x from the neutral axis, which is elongated by the amount ab by a force p; its original length being equal to AB; then by (11) we have

$$\mathrm{E} \,.\, a = p \,\frac{\mathrm{AB}}{ab}$$

$$\text{or } p = \mathrm{E} \,.\, a \,\frac{ab}{\mathrm{AB}}$$

$$= \mathrm{E} \,.\, a \,\frac{x}{\mathrm{OA}}$$

The moment of p about the neutral axis is—

$$p \,.\, x = \mathrm{E} \,.\, a. \,\frac{x^2}{\mathrm{OA}}$$

and the same for the other fibres in the section; so that the whole *moment of resistance* (M) is

$$\begin{aligned} M &= p.x + p'x' + \ldots . \\ &= \frac{E}{OA}(ax^2 + ax'^2 + \ldots .) \\ &= \frac{E}{OA} \times I \end{aligned}$$

where I is the moment of inertia of the section taken about the neutral axis; and by similar triangles,

$cd : Ac :: AB : OA$, therefore we have,

$$M = E \frac{I}{Ac} \frac{cd}{AB}.$$

But by (**11**) $E \frac{cd}{AB} = S$, the force per square inch producing the elongation cd; if then we put $Ac = z$ the distance from the neutral axis of the most extended or compressed fibre, we have

$$M = S \frac{I}{z}.$$

13. Friction.—When we attempt to slide a heavy body along the surface of any other solid material, we become conscious of a considerable resistance being offered to its motion, which it requires a certain amount of force to overcome. This resistance to motion is denominated "Friction," and is found by experiment to be proportional to the pressure which the two bodies in contact exert upon each other, but is independent (except in extreme cases) of the extent of area of the surfaces in actual contact. The resistance also depends very much on the roughness of the surfaces; for if we examine with a magnifier the surface of a piece of worked stone, which to the unaided vision appears to be smooth and level, we

shall find that it really consists of a number of hills and valleys, so that when two such surfaces are brought in contact the hills on one surface become interlocked with the valleys on the other, as in the case of toothed wheels, and it is not until the pressure which one set of hills exerts on the other has worn them both down to a level, that one body is enabled by the applied force to move over the surface of the other. Friction may, therefore, be looked upon as the *reaction* (1) of the projections upon one surface against those of the other. Hence it is that, when two bodies are rubbed together, fine particles of each substance are found to be lying loose upon their surfaces, these being the small hills above mentioned, which have been rubbed down by the pressure.

If P is the perpendicular pressure of one body on another, and we put F to represent the force of friction, then the ratio $\frac{F}{P}$ represents what is called the *coefficient of friction* for the particular surface and material, being determined experimentally for different substances in the following manner. Let the body be placed upon a plane surface, AB, which is movable about the point A, and let the plane AB be tilted up about A through an angle a until the body is just upon the point of sliding down the plane; the angle a at which sliding is about to commence is called the "angle of repose," and we find that

$$\frac{F}{P} = \tan. a,$$

for which the symbol μ is generally employed. Knowing then the value of μ or tan. a, and also of P for any par-

ticular substance, we at once get the force of friction, namely,

$$F = P.\mu = P. \tan. \alpha.$$

The value of the coefficient of friction μ for *dry* masonry and brickwork—that is to say, without a bed of mortar—is about ·65, the *angle of repose* α being 33°. For the same materials put together with mortar, the value of μ while the mortar is fresh is ·75, the angle of repose being 37°. In other words, the force required to move a piece of stone or brick along the *level* surface of a similar substance is two-thirds of its weight in the former case, and three-fourths of its weight in the latter case, when there is a bed of wet mortar interposed between them. If the mortar has been allowed to set, the force of *cohesion* (**14**) will come to the aid of friction, and the resistance will be greatly increased. Friction is one of the most important forces with which the architect is concerned, as upon it a great deal of the stability of his buildings depends.

14. COHESION is that force of attraction existing between the particles of a solid body, or between those of two substances in close contact with each other, which prevents them from being separated without the exertion of a stretching force acting in the opposite direction to the force of cohesion. In the operations of building a number of different materials are brought together in such a manner as to form one compact structure, and these are held together to a great extent by the force of cohesion. For instance, cement or mortar is employed for the purpose of uniting stones and bricks so as to form a solid wall, and in considering the stability of the structure we have to ascertain the cohesive strength not only of the

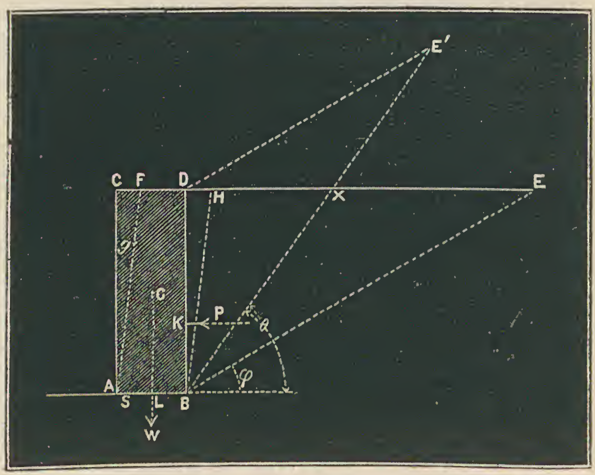
stones or bricks themselves, but also of the cement with which they are united. Now as it is a well-established rule of mechanics that the actual strength of any body or structure never exceeds the strength of its weakest part, it is of little use to have the cohesive power of the cement greater than that of the stone or brick, or to have that of the stone or brick very great when the cement which unites them has but little cohesive power. For example, the cohesive power of stone and brick varies from 300 to 500 lbs. per square inch of section; that of Portland cement, when thoroughly hardened, is about 300 lbs. per square inch when mixed with twice its bulk of sand, and 600 lbs. when used neat. The cohesive strength of cast-iron, or its resistance to tensile strain, is about 7 or 8 tons per square inch of section, while that of wrought-iron is about three times as much, and that of steel from 30 to 40 tons per inch. The cohesion of the fibres of wood is greater in the direction of their length than transversely to it, which arises from the nature of its structure; and the resistance to tensile strain is generally much greater in timber than the resistance to compression.

CHAPTER II.

RETAINING WALLS.

15. WHEN an embankment of loose earth is left to itself it assumes a gradual incline, which is called its *natural slope;* and the angle which this slope makes with the horizon is called the "angle of repose," and varies according to the adhesive qualities of the component parts

Fig. 24.



of the soil, from about 20° for wet clay to 55° for compact earth. When, therefore, earth is filled up against a vertical wall, the pressure sustained by the wall is that of the

wedge of earth which lies above the *natural slope*. Thus, in fig. 24, if AD is the section of a wall against which the earth is filled up, BE the *natural slope* making the angle ϕ with the horizontal, the wall has to support the pressure of the triangular mass of earth BDE. Let P be the pressure of the earth perpendicular to the face BD of the wall, acting at a point K, the position of which is to be hereafter determined. Let h be the height BD, t the thickness AB, supposed uniform, w the weight of a cubic foot of earth, w' that of a cubic foot of the wall. Now the earth between BE and BD may yield in any line as AX making, say, the angle θ with the horizontal, and the problem is resolved into determining the value of θ, which gives the greatest value to P. If Q is the weight of the triangular piece of earth BDE, we find, from the principles of the inclined plane, that

$$\begin{aligned} P &= Q \,.\, \tan.\, (\theta - \phi) \\ &= \tfrac{1}{2}\, wh^2.\, \cot.\, \theta \,.\, \tan.\, (\theta - \phi), \end{aligned}$$

if we take 1 foot in length of the wall. Now this quantity is found to be greatest when

$$\theta - \phi = 45^\circ - \tfrac{1}{2}\phi$$

which gives us for the value of P,

$$\begin{aligned} P &= \tfrac{1}{2}\, wh^2.\, \tan.^2 \left(45^\circ - \frac{\phi}{2}\right) \\ &= w\frac{h^2}{2} \tan.^2 \left(\frac{\text{DBE}}{2}\right). \end{aligned}$$

Referring to the case of the pressure of water against the side of a tank (**73**), we find that F being that pressure, and w the weight of a cubic foot of water,

$$F = w\frac{h^2}{2}$$

so that P may be considered as the pressure of a mass of liquid, of which the weight per cubic foot is

$$w \cdot \tan.^2\left(\frac{\mathrm{DBE}}{2}\right);$$

And as the pressure increases uniformly from the top of the wall downwards, the position of the point at which the resultant of the forces acts must be that of the centre of pressure (**74**), as in the case of water, at a distance of one-third of the height of the wall from the bottom, or

$$\mathrm{BK} = \tfrac{1}{3}\,h.$$

The force P tends to overturn the wall about its outer edge A, and the force which resists it is the weight W of the wall itself acting at G, its centre of gravity; the moments therefore of W and P, taken about A, must balance each other in order that the wall may be in equilibrium. In practice, however, it will not do for a wall to be only just in equilibrium with the forces acting on it; and to secure stability, it is usual to take the moments about a point S within the base, where $\mathrm{AS} = \frac{1}{8}t$.

The moment of P about S is $\frac{1}{3}$ P.h, and that of W about S is $\frac{3}{8}\,w'ht^2$; and if we equate the moment of P with that of W, we obtain the following formula for the thickness t of the wall:

$$t = \tfrac{2}{3}\sqrt{\frac{w}{w'}} \cdot h \cdot \tan.\frac{\mathrm{DBE}}{2}.$$

The values of $\tan.\dfrac{\mathrm{DBE}}{2}$ for different angles of repose are given in the table at the end of this chapter, as well as the corresponding values of t, as calculated from this formula.

If we take CF = 3AS, and make AF the face of the wall, we shall find that the centre of gravity (g) of this wedge is exactly over S, so that the value of the moment of its weight about S is zero; but the wall will still have the same stability or power of resisting the force P, while $\frac{3}{13}$ths of the masonry will be saved. In this case the position of G has been shifted further back, so that the lever arm SL of W is increased in the same proportion as W itself is decreased, or the moment of W about S remains unaltered.

A further increase to the resistance of the wall is obtained if we take the wedge ACF from the front of the wall and place it in the position BDH at the back, for in this case we find

$$t = \tfrac{2}{3}\sqrt{\tfrac{2}{3}}\sqrt{\frac{w}{w'}}\,h \,.\, \tan.\,\frac{\text{DBE}}{2}$$

$$= \tfrac{4}{5}\cdot\tfrac{2}{3}\sqrt{\frac{w}{w'}}\,h \,.\, \tan.\,\frac{\text{DBE}}{2}$$

Comparing this result with that obtained above, we see that the thickness may be $\frac{4}{5}$ths of that which is necessary when the wall is vertical at the back.

16. When, instead of being level at the top of the retaining wall as at DE, the earth is sloped at the angle of repose as DE′, we have to find the angle θ which BE′ makes with the horizontal when the pressure P of the triangular piece of earth BDE′ is greatest against BD. Putting Q for the weight of the earth, we have as before

$$\text{P} = \text{Q} \cdot \tan.\,(\theta - \phi)$$

$$= \tfrac{1}{2}\,wh^2 \cdot \frac{\cos.\,\theta \cdot \cos.\,\phi}{\cos.\,(\theta - \phi)}$$

which has the greatest value when $\theta = \phi$, in which case

$$P = \tfrac{1}{2}\, wh^2 \cdot \cos.^2\phi$$

and the moment of P about S is

$$\tfrac{1}{3}\, P \cdot h = \tfrac{1}{6}\, wh^3 \cdot \cos.^2\phi.$$

The moment of W is as before

$$W \times SL = \tfrac{3}{8}\, w' \cdot h \cdot t'^2$$

Equating the moment of P with that of W, we get for the thickness t' of the wall,

$$t' = \tfrac{2}{3} \sqrt{\frac{w}{w'}} \cdot h \cdot \cos.\ \phi$$

The values of cos. ϕ for different angles of repose, and the corresponding values of t', as calculated from this formula, are given in the table below.

In the following table the values of t and t' are calculated on the supposition that the retaining wall is of rectangular section and built of solid concrete whose weight per cubic foot (w') is 140lbs. Or where the wall is battered on the face as AF, t or t' will be the thickness of the wall at AB, the thickness FD at the top being $\frac{5}{8}$ AB.

Kind of Earth.	w in lbs.	ϕ	tan. $\frac{DBE}{2}$	cos	t	t'
Compact earth	126	55°	·315	·574	·20h	·36h
Dry do.	120	45°	·414	·707	·26h	·44h
Shingle . .	112	40°	·466	·766	·28h	·46h
Dry sand .	100	40°	·466	·766	·26h	·43h
Dry clay . .	120	40°	·466	·766	·29h	·47h
Wet do. .	130	20°	·700	·940	·46h	·61h
Gravel . .	110	30°	·577	·866	·34h	·51h

Those of our readers who wish to study this subject from a theoretical point of view are referred to an article in the

Philosophical Transactions for 1856, "on the stability of loose earth," by W. J. Macquorn Rankine, F.R.S., the subject of which is the mathematical theory of that kind of stability which in a mass composed of separate grains, arises wholly from the mutual friction of those grains, and not from any adhesion amongst them. The investigation is based on the following "Principle":—"The resistance to displacement by sliding along a given plane in a loose granular mass is equal to the normal pressure exerted between the parts of the mass on either side of that plane, multiplied by a specific constant." This "specific constant" is the *coefficient of friction* of the mass, and is regarded as the tangent of the *angle of repose.*

CHAPTER III.

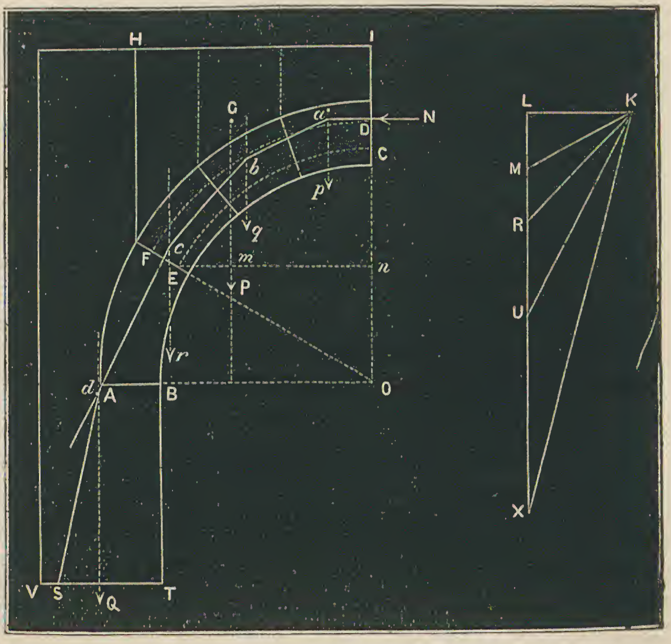
ARCHES; CUPOLAS; SPIRES.

17. The Arch, which we now propose to consider, is a structure consisting of a number of wedge-shaped stones called "voussoirs," arranged in the form of a circle or other geometrical figure, resting upon two piers, and carrying a wall or other superstructure. These voussoirs are kept in their place by their mutual pressures which are brought into play by their own weights, together with that of the superstructure, and by the resistance offered by the piers or abutments. If a straight line is drawn through the bed joint of each voussoir which shall represent the resultant of the pressures upon that joint, a polygon will be formed by the lines representing the several resultants; the curve which touches all the sides of the polygon is the "line of pressures" of the arch, and the points where it cuts the joints are the "centres of resistance." If we divide the depth of the joints into four equal parts, the *centres of resistance* ought to lie in the two *middle* divisions, in order that the joint may not have any tendency to open either at its outer or inner edge. Also the *line of pressures* must fall within the thickness of the piers. The depth of the voussoirs also should not be less than one-thirtieth of the span in a semi-circular arch, otherwise the arch will give way by the opening of the joints. It is also desirable that the mean pressure upon

any joint should not exceed one-twentieth of the ultimate resisting power of the material composing the voussoirs.

If a transverse section is made of a cylindrical arch, the inner curve is called the "intrados," and the outer curve the "extrados."

Fig. 25. Fig. 26.



The following example will explain more fully what has been just stated, and will serve to elucidate the principles upon which the stability of an arch depends. Let ABCD (fig. 25) represent the section of half a semi-circular arch carrying a wall or "surcharge," whose summit is the horizontal line HI. The arch is kept in a condition of stability by the pressure N at D of the

other half-arch, by the weight W of the portion between DC and EF and its surcharge, and by the weight Q of the abutment HV. The joint EF, which makes an angle of 30° with the horizontal, is called the "joint of rupture," for reasons hereafter explained. Let the arch between EF and CD be divided into any number of voussoirs. In the present case we divide it into only three voussoirs for the sake of simplifying the problem. Take DC and FE equal to half the depth of the voussoirs; then, as before mentioned, it is requisite that the line of pressures shall fall between the dotted lines CE and DF. Find the centre of gravity, G, of the arch ED and its surcharge, by one of the methods previously given (6) or by the method given in (19), and let P be the weight of this mass represented by the area of the section. Find also the centre of gravity of each of the portions into which the arch and surcharge are divided, and let the weights or areas of each be p, q, and r. The force N is determined from the equation

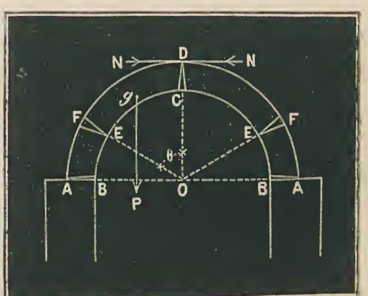
$$N = W \frac{Em}{Dn},$$

and its direction is horizontal, and meets the vertical ap from the centre of gravity of the first voussoir and surcharge in the point a. Let the horizontal line KL (fig. 26), represent N on any scale; draw the vertical LX, and take LM as the weight or area p on the same scale; then KM will be the resultant of N and p. Draw ab parallel to KM meeting a vertical from the centre of gravity of the second voussoir and surcharge in b. Take MR (fig. 26) to represent, on the same scale as before, the weight or area q, then KR will be the direction and magnitude of the resultant of N, p and q. Draw bc parallel to KR meeting the vertical from the centre of gravity of the third voussoir

and surcharge in *c*. Take RU (fig. 26) as the weight or area *r*, then KU represents the resultant of N, *p*, *q*, and *r*, or of N and W. Draw *cd* parallel to KU meeting the vertical from the centre of gravity of the pier VH in *d*, and let UX (fig. 26) be the weight Q, of the pier, on the same scale as the rest; then KX will be the resultant of all the forces N, W, and Q, both in magnitude and direction. Draw *d*S parallel to KX, then if the point S falls within the base of the pier there will be stability; but if it falls outside or beyond the edge V, the structure will be unstable and will be pushed over by the thrust of the arch. The points where the lines *ab*, *bc*, *cd*, cut the joints of the voussoirs are the *centres of resistance*, and are situated upon the line of pressures. By help of such a diagram we can determine whether any arch of given dimensions is in stable or unstable equilibrium.

18. The main object aimed at in the investigations of the arch is to find what thickness must be given to the abutments in order that they may be strong enough to resist the thrust of an arch, of which the span and depth of the voussoirs is known, as well as the height of the piers and of the superstructure.

Fig. 27.



In order to understand the method of investigation, we must first examine how an arch will break up or give way if overloaded, or if the piers are too weak to sustain it. In fig. 27 we have the section of a circular arch resting on two piers; this arch we may consider as divided into two

equal parts by the vertical line through the centre, which balance each other by their mutual resistance at CD. If this arch is overloaded or the piers are built too thin to resist the outward thrust which the voussoirs produce by their mutual pressures, the centre will sink, and the haunches will rise at EF, so that the arch will tend to break up into four parts; by the opening of the joint CD at C, of EF at F, and of the springing joint AB at B; hence it appears that when the arch is about to fall, the whole thrust at the crown, which we will call N, acts at the point D. The joint EF is called the *joint of rupture*, and is evidently that at which the effect of the thrust N is greatest, and its exact position in any particular form of arch can be determined by mathematical investigation. For if P is the weight of the portion between EF and CD, acting at g its centre of gravity, and its moment about E is $P . x$, while that of N is $N . y$, we have only to equate these moments,

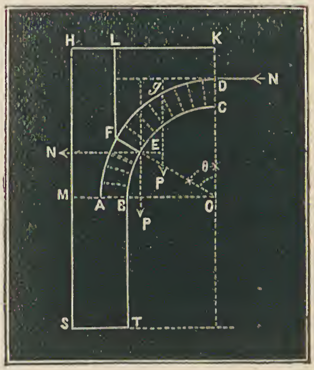
$$N . y = P . x, \text{ or } N = P\frac{x}{y},$$

and then calculate the thrust N for different values of the angle θ which FE makes with CD, and we shall find what value of θ makes N greatest.

The algebraical expression for the value of N is rather complicated, and the full investigation will be found in Captain Woodbury's "Treatise on the Arch," and Fenwick's "Mechanics of Construction." It is there shown that the greatest value of N is obtained when θ is 60°, the arch being semicircular and all the voussoirs of equal depth. When the thrust N of the opposite half-arch is exerting its greatest force upon the joint EF, it will act horizontally at the point D or vertex of the arch; and the

moments of N and P about E will balance each other. We can, therefore, by the principles of mechanics (5) transfer them to the point E, and consider them as acting there with their original values and parallel to their original directions. The forces which are now acting on the pier, are P the weight of the arch between EF and CD (fig. 28), together with its surcharge FDKL, supposed to act vertically at E; the force $N = P\,\frac{x}{y}$, supposed to be acting horizontally at E; the weight F of the portion HMBEFL acting vertically through its centre of gravity; and the weight Q of the pier MSTB, also acting vertically through its centre of gravity. Equating the moments of P, F and Q taken about S, to that of N, we shall obtain the thickness (t) which must be given to the pier in order that all the forces may be in equilibrium, or t is found from the equation,

Fig. 28.



$$N \,.\, b = P \,.\, a + F \,.\, c + Q \,.\, q \quad . \quad \text{(A)}$$

which is called the *equation of equilibrium*; a, b, c, and q, being the perpendicular distances from S of the directions of the forces P, N, F, and Q, respectively. But to secure the stability of the structure we must multiply N by the *coefficient of stability*, which we may safely take as 2. We have then to determine t from the following *equation of stability*,

$$2\,N \,.\, b = P \,.\, a + F \,.\, c + Q \,.\, q \qquad \text{(B).}$$

The following are the values of each of these four quantities in the semicircular arch one foot in width of soffit. Let δ be the weight per cubic foot of the arch and its surcharge, δ_1 that of the pier; r the radius or half-span, h the depth of the voussoirs, k the height of surcharge DK; H the height of the pier from the base S to the springing AB, t its thickness ST.

$$\mathrm{N}\,.\,b = \delta\,\frac{2\,\mathrm{H} + r}{2\,h + r}\left\{\frac{3\,k}{8}(r^2 - h^2) + \cdot026\,r^3 + \frac{3\,h}{8}r^2 - \frac{3h^2}{16}r - \frac{h^3}{4}\right\}$$

$$\mathrm{P}\,.\,a = \delta\,(\cdot134r + t)\left\{\cdot87\,k\,(r + h) + \cdot65\,(r + h)^2 - \cdot524\,r^2\right\}$$

$$\mathrm{F}\,.\,c = \delta\left\{\frac{t^2}{2}(r + h + k) + t\,(\cdot089r^2 - \cdot3h.r + \cdot134k\,.\,r - \cdot65\,h^2 - \cdot87\,h\,.\,k) + \cdot0053\,r^3 - \cdot05\,h\,.\,r^2 + \cdot009\,k\,.\,r^2 + \cdot1\,h^2r - \cdot12\,h\,.\,k\,.\,r\,. + {} + \cdot38\,k\,.\,h^2 + \cdot25\,h^3\right\}$$

$$\mathrm{Q}\,.\,q = \frac{\delta_1}{2}\,\mathrm{H}\,.\,t^2.$$

When the pier is of the same material as the arch, the quantities δ and δ_1 can be omitted altogether. If the horizontal thrust N acting at E is required, we obtain its value from

$$\mathrm{N} = \frac{2\,\delta}{2h + r}\left\{\tfrac{3}{8}\,k\,(r^2 - h^2) + \cdot026\,r^3 + \tfrac{3}{8}r^2.\,h - \frac{3h^2}{16}r - \frac{h^3}{4}\right\}$$

the value of b being $\mathrm{H} + \dfrac{r}{2}$.

These formulæ appear rather complicated, but they are easy of application, and a little practice will enable anyone who has a very moderate acquaintance with algebra to calculate them readily.

Example.—Let $r = 10$, $h = 2$, $k = 5$, $H = 10$, $\delta = \delta_1 = 120$ lbs. The value of N for an arch one foot width of soffit and of the above dimensions is 4656 lbs. acting horizontally at E. Omitting the values of δ and δ_1 from the expressions as they will divide out in forming the equations, we obtain

$$N . b = 582 \,;\ P . a = 125{\cdot}16 + 93{\cdot}4\, t. \,;$$
$$F . c = 8{\cdot}5\, t^2 - 1{\cdot}7\, t + 1{\cdot}4 \,;\ Q . q = 5\, t^2.$$

And the equation (A) therefore becomes

$$13{\cdot}5\, t^2 + 91{\cdot}7 t - 455{\cdot}4 = 0,$$

which gives $t = 3\frac{1}{3}$ ft. for equilibrium.

The equation (B) for stability is

$$13{\cdot}5\, t^2 + 92\, t - 1037{\cdot}4 = 0$$

from which we find $t = 6$ft. for stability.

In a semicircular arch of any other span, but of which the several proportions are the same as in the foregoing example, the thickness of the pier can be determined by multiplying the radius or half span by ·6 for stability, or by ·33 for equilibrium. If the height of the pier in this example is 100 instead of 10, the value of t will be 10·9 for stability; and in any other similar structure of like proportions, the thickness of the pier can be found by multiplying the radius by 1·09. In arches of different span, but having the several parts in the *same proportion*, the thrust N varies as the *square* of the span; so that if the span is doubled the thrust is increased fourfold.

If in the example above given we make $k = 8$, and H $= 20$, then the value of t is found to be $5\frac{1}{4}$ft. for equilibrium, and $8\frac{2}{3}$ft. for stability, the mean of which is about one-third the span of the arch. From this we obtain a general rule, that if the height of the pier to the springing of the arch, and the height from the springing line to the top of the surcharge, are each equal to the span of the arch, then the thickness of the abutments should not be less than one-third of the span of the arch.

In all the above calculations we have taken it for granted that the materials composing the arch, pier and surcharge have the same specific gravity or weight per cubic foot. If, however, the pier is of a different weight per foot from the rest of the structure, the values of δ and δ_1 must be included in the expressions; when, however, the specific gravity of the surcharge differs from that of the arch the formulæ become much more intricate, but the difference in the results obtainable will be only slight and may generally be neglected.

19. The thrust of an arch and the requisite thickness of pier may be obtained approximately by the following geometrical process, published by the present author in "The Builder" for December 28, 1867. Let there be first drawn carefully to scale a section of the half arch with its pier and surcharge, as in fig. 29, making the arc AF one-third of the arch AD.

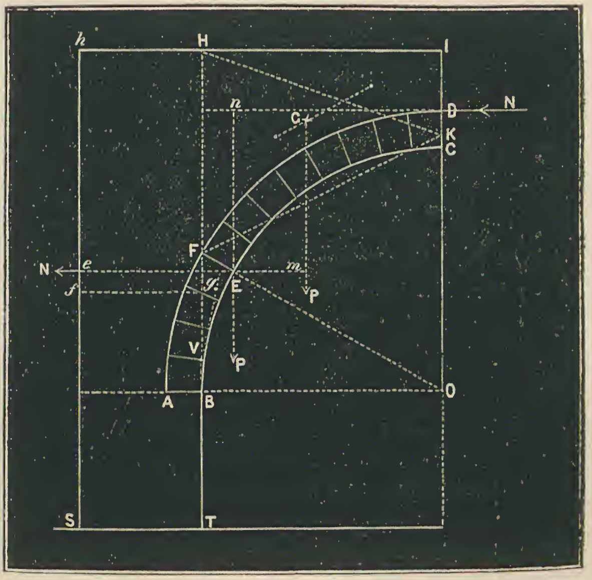
We must now find the position of G, the centre of gravity of the portion HFECI, which can be done very nearly by taking CK $= \frac{1}{3}$ CD and joining FK; then the trapezium HFKI will have its centre of gravity corresponding very nearly with the point G, and its position is found by the method given in (6) fig. 10, by dividing the trapezium into two triangles by the line HK. We have then P the

weight of HFECI acting at G, and its moment about E is P × Em; N the thrust of the other half-arch acting at D, has a moment about E of N × En (Em and En being measured by scale). Therefore $N = P\frac{Em}{En}$; P being represented by the area of the trapezium, namely,

$$\frac{HF + IK}{2} \times HI.$$

The force Q we may take as the area of the whole figure HS acting at its centre of gravity, and F as that

Fig. 29.



of a triangle VFE also acting at its centre of gravity (g) determined as shown in (6). The forces N and P are to be considered as acting at E, and we have now to take

the moments of P, F and Q about S, and equate them to twice the moment of N taken about S; we thus obtain a quadratic equation from which to find t, the necessary thickness of the pier for stability. The moment of N is $N \times Se$; the moment of P is $P \times Ee$; the moment of F is $F \times gf$; and that of Q is $HT \times \frac{t^2}{2}$. The application of this method is best shown by an example. Let $OB = 10$, $AB = \frac{3}{2}$, $ID = \frac{5}{2}$, $BT = 10$; then we find by measurement that $HF = 8{\cdot}25$, $IK = 3{\cdot}5$, $HI = 10$, $Em = 3$, $En = 6{\cdot}5$, Se or $b = 15$, $Ee = t + 1{\cdot}3$, $fg = t + {\cdot}44$, $VF = 4$. Therefore

$$P = \frac{HF + IK}{2} \times HI = \frac{8{\cdot}25 + 3{\cdot}5}{2} \times 10 = 58{\cdot}75.$$

$$N = P\frac{Em}{En} = 58{\cdot}75 \times \frac{3}{6{\cdot}5} = 27{\cdot}1.$$

To find the actual value of N for an arch of one foot length of soffit, or the horizontal thrust at E, we must multiply the above quantity by the weight of one cubic foot of the material used in the structure; and if that is 120 lbs., the horizontal thrust of the arch at E will be $27{\cdot}1 \times 120$ or 3252 lbs.

We can now form the moments of the forces about S as in the previous article (**18**).

$$N \,.\, b = 27{\cdot}1 \times 15 = 406{\cdot}5,$$

$$P \times Ee = 58{\cdot}75\,(t + 1{\cdot}3) = 58{\cdot}75t + 76{\cdot}4,$$

$$F \times fg = \frac{4 \times 1{\cdot}3}{2}\,(t + {\cdot}44) = 2{\cdot}6\,t + 1{\cdot}14,$$

$$Q\frac{t}{2} = HT\frac{t^2}{2} = 12t^2.$$

The equation for determining the value of t for stability is formed by putting $2\,\mathrm{N} \,.\, b$ equal to all the rest, which gives,

$$813 = 12t^2 + 61{\cdot}35\,t + 78{\cdot}94,$$
$$\text{or, } t^2 + 5{\cdot}1\,t - 61{\cdot}2 = 0,$$

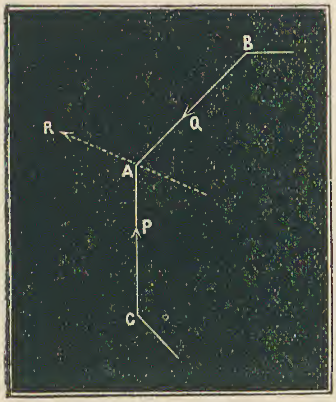
from which we obtain $t = 5{\cdot}7$.

The foregoing method will also enable us to determine whether a given thickness of pier (t) will suffice to secure the stability of an arch of known dimensions. In the last example, let us suppose 5 feet to be the given value of t, then we must put 5 for t in all the quantities which contain t, and see whether their sum is less or greater than 813, which is the value of $2\,\mathrm{N}\,b$; if it is less, then 5 is too little, and if greater, it is more than enough. When $t = 5$, $\mathrm{P} \times \mathrm{E}e = 370$, $\mathrm{F} \times fg = 14{\cdot}4$, $\frac{\mathrm{Q}t}{2} = 300$; the sum of these is 684, which being less than 813, the structure will be wanting in stability. If we take $t = 6$, then we find their sum is 877, which being more than 813, the structure will possess ample stability. We can thus by trials of two or three values of t find the thickness that will ensure stability, without having to solve a quadratic equation.

20. When several arches of equal size are arranged in a straight line and supported on piers, their thrusts will balance each other, so that the piers need not be of the strength which we have been finding by the foregoing formulæ, but only sufficiently strong to support the superincumbent load acting vertically upon them; and it is only necessary that the end piers or abutments shall be of the strength given by the above rules. When, however, the arches are arranged round a circle or polygon

instead of in a straight line, the case is different, there being an outward thrust which is not counterbalanced by the mutual reactions. For example, suppose we have eight arches arranged in the form of an octagon on plan and supported only upon light piers; let AB (fig. 30) represent one side of the octagon; AC an adjoining side; P the horizontal thrust of the arch AC, Q that of the arch AB; the forces P and Q being equal will, by the principles of mechanics (2), have a resultant R whose direction bisects the angle BAC. The tendency therefore of the forces P and Q is to thrust out A in the direction, AR; and we must therefore have a counterbalancing force equal to R acting towards A in the direction RA, in order that the forces may be in equilibrium. From (2) we at once find the relation which R must bear to P and Q;

Fig. 30.



$$\text{for, } R : P :: \sin. QAP : \sin. RAQ,$$

$$\text{or, } R : P :: \sin. 45^\circ : \sin. 67^\circ\tfrac{1}{2}$$

$$:: 101 : 132$$

so that $$R = P \times \frac{101}{132}.$$

Example :—Let eight arches be arranged in the form of an octagon, each being of the form and dimensions of the example in (19). Let the soffit of the arches be

2 feet wide, then the value of P or of Q is 2×3252 or 6504 lbs.

Therefore $\quad R = 6504 \dfrac{101}{132} = 4976$ lbs.

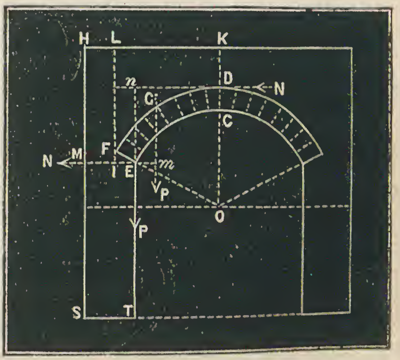
The force R may be the thrust of another arch placed in the direction AR; and if this arch has the same proportions as the other arches, we can at once determine its span, since the thrusts of similar arches are to one another as the squares of their spans. Now the span of the arches forming the octagon being 20 feet, we find,

$$\text{span of arch in AR} = 20 \sqrt{\frac{R}{P}} = 20 \sqrt{\frac{101}{132}}$$
$$= 17\cdot5 \text{ feet.}$$

No allowance is here made for the resistance offered by the pier, which is supposed to be only of sufficient strength to bear the vertical weight of the superstructure.

21. Segmental Arch.—The thrust of segmental arches, and the requisite thickness of pier, may be determined in the same manner as in semicircular arches, with such modifications as the particular case may demand. Let us take an arch whose form is that of a segment of a circle, subtending 120° at the centre, as in fig. 31. The joint of rupture will be at the springing or skewback EF, since FEO makes 60° with the vertical; the forces P and N will be the same as in the semicircular arch

Fig. 31.



(**18**) and will act at E; N will act in the direction of EM, and P in that of ET; P being the weight of the figure LFECK acting at its centre of gravity G; Q the weight of the pier MSTE. The force F we can now consider as the area of the parallelogram HMIL (neglecting the small wedge EFI). Then the moments of the forces about S are

$$P \times EM = P \,.\, t;\; N \times ET = P\,\frac{Em}{En}\,ET;$$

$$F \times \frac{MI}{2} = HM\,\frac{MI^2}{2};\; Q\,\frac{t}{2} = ET\,\frac{EM^2}{2} = ET\,\frac{t^2}{2}.$$

Putting ET = H, and the other dimensions as in (**18**) we have as before (page 48)

$$N \times ET = \delta\,\frac{2H}{2h + r}\left\{\frac{3h}{8}(r^2 - h^2) + \cdot 026\, r^3 + \frac{3h}{8}r^2 - \frac{3h^2}{16}r - \frac{h^3}{4}\right\}$$

$$P \,.\, t = \delta . t\{\cdot 87h\,(r + h) + \cdot 65\,(r + h)^2 - \cdot 524 r^2\}$$

$$F \times \frac{MI}{2} = \delta \,.\, HM \times \frac{MI^2}{2} = \frac{\delta}{2}\left(\frac{r}{2} + h + h\right)(t - \cdot 87h)^2$$

$$Q \times \frac{t}{2} = \delta_1 \,.\, ET \times \frac{t^2}{2} = \frac{\delta_1}{2}\,H . t^2.$$

Then the value of t for stability is determined as before, by equating twice the moment of N to the sum of the three other moments. The span in this arch will be $1{\cdot}732r$.

Let us apply it to an arch having the dimensions of the example in (**18**). The actual value of N for an arch having a soffit one foot wide is, as before, $120 \times 38{\cdot}8$, or 4656 lbs. Omitting δ, and making ET = 10, we get

$$2\,N \times ET = 776;\; P . t = 93{\cdot}4t;$$

$F \times \frac{MI}{2} = 6t^2 - 20{\cdot}88t + 18{\cdot}24$; $Q\frac{t}{2} = 5t^2$; and the equation for stability becomes

$$776 = 11t^2 + 72{\cdot}5t + 18{\cdot}24$$

which reduces to

$$t^2 + 6{\cdot}6t - 68{\cdot}9 = 0;$$

therefore $$t = 5{\cdot}8 \text{ ft.}$$

The span of this arch is 17·32 feet. In any other segmental arch of similar proportions, the thickness of pier will be $\cdot 58r$.

We will now take the case of a segmental arch which subtends an angle of 90° at the centre O. The joint of rupture will be at the springing as before, but the springing EF being now nearer the crown, the thrust N is less than in the semicircular arch of same radius. The value of N acting horizontally at E for an arch whose soffit is 1 foot in width is,

$$N = \frac{\delta}{h + \cdot 29r}\left\{\cdot 012r^3 + \cdot 25\,(h + k)\,r^2 - \cdot 07\,k^2\,r - \cdot 13k^3 - \cdot 25kh^2\right\}$$

and the moment of N about S is $N \times ET = N \times H$. Also,

$$P \times ST = \delta \,.\, t\{\cdot 7k\,(r + h) + \cdot 46\,(r + h)^2 - \cdot 4r^2\}$$

$$F \times \frac{MI}{2} = \frac{\delta}{2}(\cdot 3r + h + k)\,(t - \cdot 7h)^2$$

$$Q \times \frac{t}{2} = \frac{\delta_1}{2}\,H\,.\,t^2.$$

The span of this arch is $1{\cdot}414r$.

As an example of this form of arch, we will find the thrust and thickness of pier when the other dimensions are the same as in the last example.

The value of N for an arch having a soffit one foot wide and of material weighing 120 lbs. to the cubic foot, is $120 \times 36{\cdot}3$, or 4356 lbs., which is the horizontal thrust at E. Then, supposing the whole structure to be built of the same material, and therefore omitting the weight δ from the calculations, we have twice the moment of N about S equals 726 ;

$$P \,.\, t = 68t; \; F \times \frac{MI}{2} = 5t^2 - 14t + 9{\cdot}8\,;$$

$$Q\frac{t}{2} = 5t^2. \text{ The equation for stability is}$$

$$726 = 10\,t^2 + 54t + 9{\cdot}8\,;$$

or,

$$t^2 + 5{\cdot}4t - 71{\cdot}6\,; = 0\,;$$

$$\therefore t = 6{\cdot}2 \text{ ft.}$$

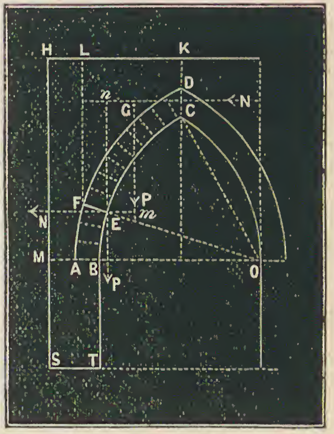
The span in this case is 14·14 ft. In any other segmental arch of the same form and proportions, the thickness of pier must be $\cdot 62r$.

If in this last example, we make the height H, of the pier ET, 100 feet instead of 10 feet, its thickness must be 11 feet ; and in the like proportion for any other height between 10 and 100 feet.

The *flat-arch* frequently used over small openings is in reality a segmental arch, as the joints are made to radiate towards a fixed point O, the distance of which from the skewback EF may be considered as the radius r. Thus, suppose a horizontal line to be drawn across at E and another at D (fig. 31), and the line OEF produced to meet the horizontal through D, and the joints of the voussoirs of the segmental arch DCEF to be produced to meet these two horizontal lines, we shall have a *flat-arch* represented by the segmental arch, and its thrust on the wall will be the same as that of the segmental arch.

22. Gothic Arch.—The thrust of "Gothic" or pointed arches (fig. 32) can be calculated in the same manner as described for semicircular arches, only the position of EF, the joint of rupture, has to be determined for each particular case separately, as it varies according to the *pitch* of the arch. Thus, in the equilateral arch, in which OC makes 30° with the vertical, the joint FE makes the angle 16° with the horizontal; in an arch in which OC makes 15° with the vertical, EF makes 28° with the horizontal; and when OC makes 45° with the vertical, EF makes 10° with the horizontal. The equilateral arch may be taken as a good representation of the general forms of large Gothic arches.

Fig. 32.



In this case we suppose N to act at the middle of the joint CD, its moment about E being N × En which equals P × Em; P being, as before, the weight of the portion LFECK acting at G its centre of gravity. Also F is the weight of the part HLFEBM acting at its centre of gravity; Q the weight of the pier MBTS. Then the value of N acting horizontally at E is found to be in the equilateral arch,

$$N = \frac{\delta}{\cdot 6r + \cdot 6h} \left\{ h\,(\cdot 11r^2 - \cdot 46h^2) + \cdot 013r^3 + \cdot 12r^2 \,.\, h - \cdot 27h^2 \,.\, r - \cdot 45h^3 \right\}.$$

If this is multiplied by the height SN, or b, we have N . b

the moment of N about S. In this case $b = \mathrm{H} + \cdot28r$. The moments of P, F and Q about S are

$$\mathrm{P}.a = \delta\,(\cdot04r + t\,)\left\{k\,(\cdot46r + h) + \cdot1r^2 + hr + h^2\right\}$$

$$\mathrm{Q}\,.\,q = \frac{\delta_1}{2}\,\mathrm{H}\,.\,t^2$$

$$\begin{aligned}\mathrm{F}\,.\,c = \delta\,\Big\{&t^2\,(\cdot5k + \cdot43\,r + \cdot6h) + \\ &+ t\,(\cdot026r^2 + \cdot04\,k\,.\,r - \cdot52h\,.\,r - k^2 - h\,.\,k) \\ &+ k\,(\cdot008r^2 + \cdot04\,h\,.\,r + \cdot5k^2) \\ &+ \cdot006r^3 + r\,.\,h\,(\cdot24h - \cdot013r) + \cdot45k^3\Big\}.\end{aligned}$$

These are readily calculated by putting for r, h, k and H, their respective values; $r = \mathrm{OB}$, $h = \mathrm{EF}$ or AB, $k = \mathrm{DK}$, $\mathrm{H} = \mathrm{MS}$. We then find t for stability from the equation,

$$2\,\mathrm{N}\,.\,b = \mathrm{P}\,.\,a + \mathrm{F}\,.\,c + \mathrm{Q}\,.\,q.$$

Example:—Let $r = 10$, $h = 1$, $k = 4$, $\mathrm{H} = 10$, $\delta = \delta_1 = 120$ and $b = 12\cdot8$. Then $\mathrm{N} = \delta\,.\,9\cdot7$, or omitting the values of δ and δ_1 we have $2\,\mathrm{N}\,.\,b = 248\cdot3$; $\mathrm{P}\,.\,a = 43\cdot4t + 17\cdot4$; $\mathrm{F}\,.\,c = 6\cdot9t^2 - 6t + 11\cdot15$; $\mathrm{Q}\,.\,q = 5t^2$. Therefore the equation is,

$$248\cdot3 = 11\cdot9t^2 + 37\cdot4t + 28\cdot55;$$
$$\text{or, } t^2 + 3\cdot14t - 18\cdot5 = 0;$$
$$\therefore t = 3 \text{ feet.}$$

The horizontal thrust of N at E in this arch for every foot width of soffit is 1164 lbs. In any arch of similar proportions the thickness of the pier must be three-tenths of the span. If we increase the height of the pier to 100 feet, or ten times the span, the thickness of pier must be 5·55 feet, or $\cdot555r$. These investigations of the

thrust of Gothic arches were first published by the present writer in "The Builder," March 31, 1866.

23. Elliptic Arch.—The horizontal thrust of an elliptical arch can be found approximately from that of a semicircular one; being nearly the same as that of a semicircular arch, having the same span, but whose vertical depth is to that of the elliptical arch as the major to the minor axis of the ellipse. (Fenwick's "Mechanics of Construction.") Thus, suppose the height of the ellipse is 20 feet to a span of 60 feet, the axes being in the proportion of 3 to 2; then if the voussoirs at the crown are 2 feet deep, the equivalent circular arch will have a radius of 30 feet, and depth of voussoirs 3 feet. This is only an approximation, and the exact relations between the several parts may be investigated directly in the same manner as given for the circular arch; but the algebraical formulæ are more complicated, and the position of the joint of rupture must be determined in each particular case. In an ellipse, however, which is not very flat, the joint of rupture may be taken as very nearly that which makes 30° with the horizontal, as in the circular arch.

24. Gothic Vaulting consists of a number of stone arched ribs which spring from the walls of a building, or from corbels projecting therefrom; the space between the ribs being filled in with stone panelling of a light character. To avoid complicating the question of thrust in such arches, we will suppose the vaulting to consist of transverse ribs whose direction is at right angles to the walls, and diagonal ribs making 30° with the former, as in the plan, fig. 33. The transverse rib (fig. 34) we will suppose to be an equilateral arch. We will at present omit from the investigation all intermediate ribs and

filling in, which can be allowed for by adding something to the thickness of the main ribs themselves. The back of the ribs is filled up to the haunches as far as HK, so as to prevent them from yielding at that part; OH making 30° with the horizontal. In this kind of arch, having but little surcharge, there is a tendency to break up by the rising of the crown and the falling in of the

Fig. 33.

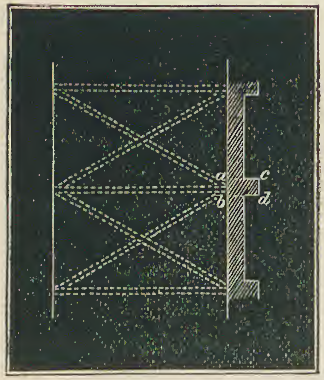
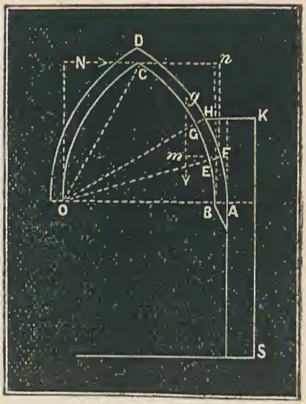


Fig. 34.



haunches; so that the joint CD will tend to open at D, and the joint of rupture EF to open at E; the portion above EF tending to turn over on the point F. We shall here take EF as making 16° with the horizontal as in (22). The horizontal thrust N at the crown will now act through C, and its moment about F will equal that of P about F; P being the weight of the part above EF acting at the centre of gravity g. Let l be the breadth of soffit of each rib, h the depth of the ribs, r the radius OB, δ the weight per cubic foot. Then the value of N is found to be

$$N = \frac{l \,.\, \delta}{\cdot 6r - \cdot 3h} \left\{ \cdot 15h \,.\, r^2 + \cdot 65h^2 \,.\, r + \cdot 4h^3 \right\}.$$

The moment of N about S, the outer edge of the base of the pier, is $N \,.\, b$; where $b = H + \cdot 28\,(r + h)$; H being the height of the pier to the springing AB. Also if we put P, the weight of the portion above EF, acting in this case at F, R that of the part HFEBA, acting vertically through its centre of gravity, their moments about S are

$$P \,.\, a = l \,.\, \delta\,(\cdot 04\,(r + h) + t) \left\{ \cdot 01r^2 + \cdot 8h \,.\, r + \cdot 7h^2 \right\}$$

$$R \,.\, c = l \,.\, \delta \left\{ t\,(\cdot 01r^2 + \cdot 3h \,.\, r + \cdot 15h^2) + \cdot 0002r^3 + \cdot 004h \,.\, r^2 + \cdot 14h^2 \,.\, r + \cdot 05h^3 \right\}.$$

The diagonal ribs will not have the same pitch as the transverse ones, but we may, without materially affecting the calculations, treat them as equilateral, having for their radius $1 \cdot 16r$. The thrust of these ribs will be oblique, so that we must only take the resolved part of the horizontal thrust N′ perpendicular to the wall, multiplying by cos. 30°, or ·87. The value of N′ acting perpendicularly to the wall for each diagonal rib is

$$N' = \frac{l \,.\, \delta}{\cdot 8r - \cdot 3h} \left\{ \cdot 2h \,.\, r^2 + \cdot 8h^2 \,.\, r + \cdot 4h^3 \right\}.$$

r being the radius of the *transverse* rib. Then the moment of N′ about S is $N' \,.\, b'$, where $b' = H + \cdot 32r + \cdot 28h$. Also we find,

$$P'.a' = l \,.\, \delta\,(\cdot 04r + \cdot 03h + t) \left\{ \cdot 013r^2 + \cdot 9hr + \cdot 7h^2 \right\}.$$

$$R' \,.\, c' = l \,.\, \delta \left\{ t\,(\cdot 013r^2 + \cdot 35h \,.\, r + \cdot 15h^2) + \cdot 0002r^3 + + h \,.\, r\,(\cdot 005r + \cdot 14h) + \cdot 04h^3 \right\}.$$

We will take the width ab (fig. 33) of the pier as $3\,l$; then we have

$$Q \, . \, q = \frac{3\,l\,.\,\delta_1}{2}\left(H + \tfrac{1}{2}(r + h)\right)t^2.$$

The equation from which t is found for stability is $2\,N\,.\,b + 4\,N'\,.\,b' = P\,.\,a + 2\,P'\,.\,a' + Q\,.\,q + R\,.\,c + 2\,R'\,c'$.

Example:—$r = 20$, $h = 1$, $l = \frac{1}{2}$, $H = 30$, $\delta = \delta_1 = 120$; $N = \delta \times 3{\cdot}14$, $N' = \delta \times 3{\cdot}07$; $b = 35{\cdot}88$; $b' = 36{\cdot}68$. The horizontal thrust N of the transverse rib at F is found to be 377 lbs.; that of the two diagonals resolved perpendicular to the wall, or $2\,N'$, is 737 lbs.; or the total thrust at F is 1114 lbs. The thickness (t) of the pier is found by the formulæ: (omitting the value of δ)

$2\,N\,.\,b = 225$; $4\,N'\,.\,b' = 450$;
$P\,.\,a = 8{\cdot}7 + 10{\cdot}4t$; $2\,P'\,.\,a' = 19{\cdot}9 + 24t$;
$R\,.\,c = 3 + 5{\cdot}2t$; $2\,R'\,.\,c' = 6{\cdot}4 + 12{\cdot}4t$;
$Q\,.\,q = 30{\cdot}4t^2$. And the equation for stability becomes

$$675 = 30{\cdot}4t^2 + 52t + 38;$$

or,

$$t^2 + 1{\cdot}7t - 21 = 0;$$

$$\therefore t = 3{\cdot}8 \text{ feet.}$$

In any other vaulting of the same proportions, t is found by multiplying the *half-span* by ·38. If the value of H is 60 feet, the value of t in the above example must be 4·3 feet.

The foregoing example applies only to a vaulting of skeleton ribs, and omits the pressure upon them which is produced by the filling in of the panels between the ribs. Let us now suppose this filling in to be 3 inches or $\frac{1}{4}$ ft. in thickness, and to be of the same kind of material as the ribs. The pressure on the transverse rib will be

greatest at the crown, and will diminish to nothing at the springing; it will be the same as if we added 2AB or $2h$ to the thickness at C, or of a rib whose depth at C is 3 feet and at AB 1 foot. The centre of gravity g at which P acts will also be moved nearer to C, so that not only is P increased but also the length of Fm its lever arm about F. We shall therefore obtain an approximate value of N if we multiply P by 2 and Em by $\frac{3}{2}$, so that in this case we have (18)

$$N \cdot y = 2P \times \tfrac{3}{2}x,$$

$$\text{or, } N = 3P\frac{x}{y},$$

or N is three times as great as in the case of the skeleton ribs. P . a will also be twice as great, so that we must put 3N for N and 2P for P in the above formulæ of calculation. In the diagonal rib the load at the crown is half that upon the transverse rib, but increases to the same amount at the springing. We may therefore put 2N′ for N′ and 2P′ for P′ in the above formulæ. We have then

$$3N = 1131\,;\ 4N' = 1474\,;$$

horizontal thrust at F $= 3N + 4N' = 2605$ lbs.

$$6N \cdot b = 675\,;\ 8N' \cdot b' = 900,$$

$$2\,P \cdot a = 17{\cdot}4 + 20{\cdot}8t\,;\ 4P' \cdot a' = 39{\cdot}8 + 48t.$$

Hence the equation for stability becomes

$$30t^2 + 86t - 1508 = 0,$$

$$\text{or, } t = 5{\cdot}8 \text{ feet.}$$

And in any other vaulting of which the several proportions are the same, the value of t is found by multiplying the *half-span* by ·58.

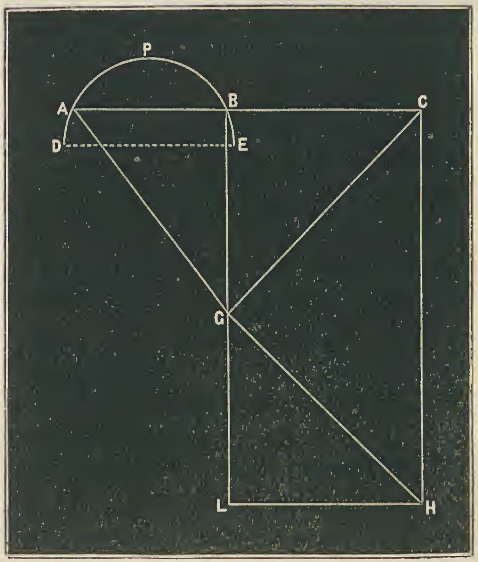
25. Oblique Arch.—When two roads at different levels cross one another in directions which are not at right angles to each other, and an arch is thrown across the lower road to carry the upper one, it is called an "oblique" or "skew" arch; and the angle which a perpendicular to the axis of the lower road makes with the axis of the upper one is called the "angle of obliquity." If the courses of an *oblique* arch were built so as to be at right angles to the abutments, as in the ordinary arch, a large part of them would have no abutment at one side, and the arch would fail. If, on the other hand, the beds of the courses were parallel to the two faces of the arch, they would abut obliquely on the piers, and the soffit would not form an even surface.

This last plan has occasionally been adopted in small arches of moderate obliquity, but would be very deficient in stability if used for large arches, or where the obliquity was considerable. The method employed in forming large oblique arches is to consider the arch as a portion of two concentric cylinders cut obliquely by the two planes forming its faces, and which are parallel to the direction of the upper roadway. The solid contained between these two cylinders is cut by a number of screw surfaces all of similar form and described about a common axis; and these again are intersected at right angles by another set of screw surfaces, also of similar form. In this way the solid contained between the two cylinders is divided into a number of equal and similar solids which are the voussoirs of the arch. The templates for forming these voussoirs require to be made with considerable accuracy, the several angles being calculated by the help of trigonometry for each particular case. The rules for getting out the templates are given in Buck's "Treatise on

Oblique Arches,"* to the last edition of which Mr. Barlow has added a method of arriving at the necessary results without the actual use of trigonometrical formulæ. A small work by Mr. W. Donaldson also gives a very accurate method. Oblique arches should always be segments of circles of considerable flatness, and the obliquity may be made with safety as great as 65°.

In fig. 35, DEP represents the semicircle of which APB is the segment employed as the section of the soffit

Fig. 35.

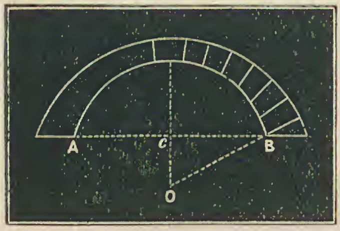


of the oblique arch. Let BAG be the angle of obliquity; BC the length of the arc APB developed, and drawn perpendicular to BG. Then GC is the spiral of the heading joints, and is to be divided into the determined

* Lockwood & Co.

number of voussoirs. To this the direction of the coursing joints is to be perpendicular. Make GH perpendicular to GC, and CH parallel to BG; draw HL perpendicular to CH; then GH is the development of the segmental spiral, and GL its axial length. The lines of the faces of the arch are ellipses, and the joints of the voussoirs all radiate from a point O (fig. 36) situated below the springing line or axis of the cylinder; its position being found by calculation.

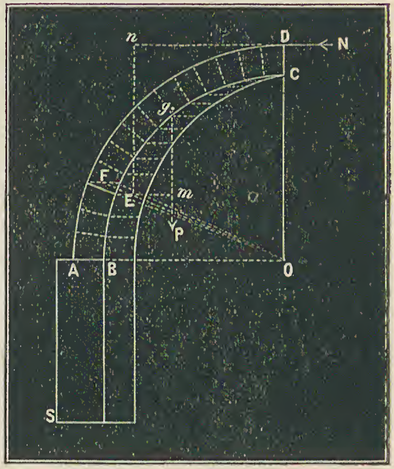
Fig. 36.



26. Cupolas.—If we suppose an arch of any form of section to revolve about its vertical axis, a hollow shell will be formed, which is called a "cupola" or "dome." Such a shell is often built over a circular chamber, either underground or above ground. When built underground, it possesses great strength and power of resistance to pressure, having all the rigidity of the vertical and horizontal arch combined, and any thrust is entirely overcome by the surrounding earth. The dome is built on the principle of the arch, having wedge-shaped voussoirs; but as it consists of a series of horizontal rather than vertical arches, it follows that as soon as any number of courses are completed the structure is in a condition of stability, and requires no further support from centering except for the construction of the remaining courses; so that any number of the upper courses may be omitted without endangering its stability. When a dome is elevated upon a circular wall or "drum," it will produce a certain horizontal thrust upon the wall. The investiga-

tion of the thrust and necessary thickness of wall may be made in the same manner as that adopted in the case of the ordinary arch, by considering the dome to consist of a number of lunes or thin slices cut out of the entire dome by two planes intersecting at the vertical axis, and making a small angle, such as 2°, with each other. Fig. 37 represents a lune cut out of one side of a circular dome, which for the sake of investigation we may consider as balanced by an equal lune cut out of the opposite side; the two lunes meeting at the crown CD, and balancing each other by their mutual pressures, which we call N, acting horizontally. Of course such lunes could not be built to stand alone, but the lateral pressure of the whole circle of lunes keeps them together. The horizontal thrust of a dome upon the walls is much less than would arise from an ordinary arch of equal span; and it may be reduced still further by means of iron belts girding the dome at a short distance above the springing. The joint of rupture in the hemispherical dome is ascertained, by analytical methods, to be that which makes 20° with the horizontal; it is therefore at this level that the iron belt will produce the greatest effect in counteracting the thrust.

Fig. 37.



Using the same notation as in the arch, we have P the weight of the part above the joint of rupture EF; F that of the part below EF; Q that of the portion of the wall or *drum* on which the lune stands; N the horizontal thrust. Having determined the position of the joint of rupture EF, we can suppose P and N to be transferred to E; F to act at the centre of gravity of the part between EF and AB; Q at the centre of gravity of the pier BS. Let R and r be the external and internal radii of the dome; δ the weight of a cubic foot of the dome. Then the thrust N at E of the 180th part of the dome is found to be,

$$N = \delta \frac{\cdot 00719 r (R^3 - r^3) - \cdot 00393 (R^4 - r^4)}{R - \cdot 342 r}.$$

Hence it appears that in domes of similar proportions the horizontal thrust varies as the *cube* of the diameter or span.

The moment of N about the outer edge of the pier is N . b, where $b = H + \cdot 342 r$; H being the height of the pier to the springing of the dome.

In order, however, to simplify the problem and render the calculation easier, we shall take the lune of the dome at the base as 1 foot in length instead of the 180th part of the whole circumference, the length of 1 foot being taken half-way between A and B; so that we must multiply the above value of N by 180, and divide by the circumference of the circle whose radius is $\frac{1}{2}(R + r)$. We shall also consider that the same material is used throughout in the construction both of the dome and of the pier, so that we may omit the value of δ from the equations. We thus obtain the following moments of the forces N, P, F, and Q, taken about the outer edge S of the pier.

$$N \cdot b = \frac{H + \cdot 342r}{R + r} \cdot \frac{\cdot 412\,(R^3 - r^3)\,r - \cdot 225\,(R^4 - r^4)}{R - \cdot 342r}$$

$$P \cdot a = \frac{R^3 - r^3}{R + r}\,(\cdot 439t + \cdot 02\dot{6}r)$$

$$F \cdot c = \frac{\cdot 149\,(R^3 - r^3)\,(t + r) - \cdot 11\,(R^4 - r^4)}{R + r}$$

$$Q \cdot q = \tfrac{1}{2}\,H \cdot t^2.$$

Then the value of t for equilibrium is obtained as in the case of the arch, from the equation

$$N \cdot b = P \cdot a + F \cdot c + Q \cdot q.$$

For stability we get t from the equation

$$2\,N \cdot b = P \cdot a + F \cdot c + Q \cdot q.$$

As an example, let $R = 11$, $r = 10$, $H = 50$; then we find $N \cdot b = 107{\cdot}2$; $P \cdot a = 6{\cdot}92t + 4{\cdot}16$; $F \cdot c = 2{\cdot}35t - {\cdot}8$; $Q \cdot q = 25t^2$. The equation for equilibrium therefore is

$$25t^2 + 9{\cdot}3t - 104 = 0,$$

which gives, $t = 1{\cdot}86.$

The equation for stability is

$$25t^2 + 9{\cdot}3t - 211 = 0,$$

which gives, $t = 2{\cdot}7.$

The value of N or the horizontal thrust at E, for the 180th part of the dome in the above example is 92 lbs. as obtained from the first equation given above, if we put $\delta = 125$ lbs.; or the total horizontal thrust is 16560 lbs. for the whole dome.

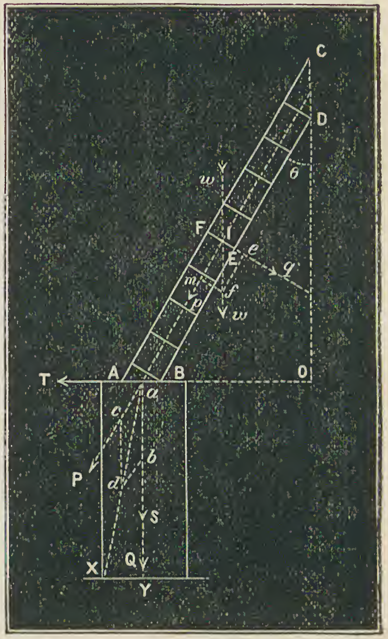
In any other hemispherical dome whose thickness is one-tenth of the radius, and the height of the pier five times the radius, the thickness of the pier for stability is $\cdot 27r$. When H is 10 ft., t is found to be 2·37 ft.; and

when H is 100 ft., t is 2·8 ft. The dome in this case is supposed to be of uniform thickness throughout, and the wall or pier of solid construction.

When the section of the dome is a Gothic or pointed arch, the position of the joint of rupture will depend on the *pitch* of the arch; if the *pitch* is low or the section differs but little from a semicircle, the joint of rupture will make an angle of nearly 20° with the horizontal, but if the *pitch* is high, it makes a much smaller angle. Thus, for instance, if the radius drawn to the vertex makes only 10° with the vertical, the joint of rupture makes 17° with the horizontal; but when that radius makes $22\frac{1}{2}$° with the vertical, the joint of rupture makes $13\frac{1}{2}$° with the horizontal; and if it is at 30° with the vertical, then the joint of rupture is 10° only with the horizontal. The value of N, or the horizontal thrust at E, diminishes as the pitch increases; so that, if in the hemisphere it is 92, in the Gothic domes of the three sections above described it is 88·7, 80·4, and 77·3 respectively, the *span* being the same in all. In a parabolic dome of the same span as the above, the height being equal to half the span, the thrust is 79·6, and acts at the springing, which is the joint of rupture. If the thickness of the hemispherical dome is made to diminish upwards, so as to be half the thickness at the crown, that it is at the springing, the thrust is greatly decreased, being only 55·8 in the dome of 20 ft. span, as compared with 92 in the dome of uniform thickness. The foregoing description is taken from two papers by the present writer on the "Stability of Domes," read before the Royal Society, May 31 and November 22, 1866, and published in the "Proceedings of the Royal Society," No. 85, and also in the "Civil Engineer and Architect's Journal" for February and March, 1868.

27. Conical Domes or Spires.—The strongest kind of dome is that which has the conical form, and is generated by the revolution of two triangles such as AOC, BOD (fig. 38) about a vertical axis OC; the part ABDC between the triangles forming the solid shell of the dome or spire. In constructing a spire the beds of the masonry should be at right angles to the slope of the cone, and not horizontal, as is frequently the case. Take any one of the beds, as EF, and let the weight w of the portion above EF act at its middle point I and be represented on

Fig. 38.



any scale by the vertical line If. By drawing a rectangle having If for its diagonal we can now resolve w into two other forces (2), namely q acting parallel to FE and represented by the line Ie, which will be counteracted by a corresponding force from the opposite side of the cone; and p acting parallel to AC and represented by the line Im or ef, which is the only force of which we need take account; and by the principle of the resolution of forces we have

$$p = w \cdot \cos \text{BDO}.$$

Suppose the spire to rest on the level top AB of the vertical wall AX, and let W be the total weight of a slice of the cone whose base is 1 foot in length measured at the middle point a of AB; let P be the resolved part of W acting at a. Then if we put θ for the angle BDO, we have,

$$P = W \, . \cos \theta$$

$$W = \frac{\delta}{3} \cdot \frac{R^2 K - r^2 k}{R + r}$$

where $OC = K$, $OD = k$, and δ is the weight of a cubic foot of the structure.

Draw the vertical line aY which we will suppose to pass through the centre of gravity of the wall. Take ac to represent on any convenient scale the force P in the direction of Ia produced, and ab on the same scale to represent the weight Q of the wall in the line aY; and complete the parallelogram $abdc$; then the diagonal ad will represent on the same scale the resultant of the forces acting on the wall, and if this line cuts the base of the wall within the outer edge X, the structure will be in a condition of stability; if it passes through the point X, it will be just in equilibrium and the least additional thrust will overturn it. If ad produced cut the base outside the point X, then the structure will be in a condition of instability, and will be overthrown.

Resolving the force P at a horizontally and vertically, and putting T for its horizontal component or the thrust at AB acting outwards and tending to overturn the wall; S for its vertical component acting in the line aY, Q for the weight or area of section of the wall also acting in the line aY; we have,

$$T = P \,.\, \sin\theta = \tfrac{1}{2} W \,.\, \sin 2\,\theta$$
$$S = W \,.\, \cos^2\theta$$
$$Q = \delta \,.\, H \,.\, t$$

where H is the height aY and t the thickness of the wall. In order to find the value to be given to t for any known value of T, we must equate the moment of T taken about X with the sum of the moments of S and Q about the same point. If we suppose the material used in the construction of the spire and the wall to be of the same density, we can omit δ from the expressions.

$$T \times AX = \tfrac{1}{2} W \cdot H \cdot \sin 2\,\theta$$
$$= \tfrac{1}{6} \frac{R^2K - r^2k}{R + r} H \,.\, \sin 2\,\theta$$
$$S \times YX = \tfrac{1}{2} W \cdot t \cdot \cos^2\theta$$
$$= \tfrac{1}{6} \frac{R^2K - r^2k}{R + r}\, t \cdot \cos^2\theta$$
$$Q \times YX = \tfrac{1}{2} H \cdot t_2.$$

The value of t which will produce equilibrium between the forces is found from the equation

$$T \times AX = (S + Q)\, YX$$

and the value of t which is required for stability in the structure is obtained from the equation

$$2\,T \times AX = (S + Q)\, YX.$$

As an example we will take the case of a conical spire whose section is an equilateral triangle; then $\theta = 30°$, $\sin\theta = {\cdot}5$, $\cos\theta = {\cdot}87$, $\cos^2\theta = {\cdot}75$, $\sin 2\,\theta = {\cdot}87$. Let $R = 11$, $r = 10$; then $K = 19$, $k = 17{\cdot}3$; and let $H = 20$. Then we find $W = 9$; $T \times AX = 78$;

$S \times YX = \frac{27}{8} t$; $Q \times YX = 10\ t^2$. The equation for equilibrium becomes

$$80\ t^2 + 27\ t - 624 = 0$$
$$\text{or, } t = 2{\cdot}63.$$

The equation for stability is

$$80\ t^2 + 27\ t - 1248 = 0$$
$$\text{or, } t = 3{\cdot}78.$$

In this form of dome or spire there is no outward thrust in the cone itself provided the beds of the stones are laid at right angles to the slope; but if the beds are horizontal there will be a tendency to slide outwards at each layer of stones, as shown by the horizontal thrust T where the base of the cone rests upon a level bed on the top of the wall at AB.

28. Iron Domes.—The domes which we have hitherto considered are supposed to be built up of masonry or brickwork, and to be of considerable thickness: these have but little lateral or cohesive strength, their stability mainly depending upon their weight and resistance to compression. There is another material, however, of which domes may in some cases be constructed, namely, wrought iron in thin plates riveted together into the form of a cone. Such a cone possesses great lateral or tensile strength which entirely prevents any outward thrust from being thrown upon the supporting wall, and its crushing strength is so great that a considerable saving in weight or pressure upon the foundations can be effected, the thickness of the plates being diminished from the base upwards, since the compression is greatest at the lowest part and decreases towards the top. The tensile strain at any part is also proportional to the diameter and therefore to the quantity of metal, being

greatest at the base where there is most resistance, and diminishing upwards with the diminution of the thickness of metal. In such a dome there is no loss of power or waste of material, the greatest amount of strength being obtained with the least possible quantity of metal. Openings to an almost unlimited extent can also be made so as to form a mere skeleton dome provided that all the parts remain connected together, and the openings may be filled in with glass or other material, without seriously affecting the stability of the structure.

Conical domes of wrought iron can be erected of much vaster dimensions, and at much less cost, than is possible with those of stone or brick, one having been built by Mr. J. Scott Russell for the Vienna Exhibition of 1873 with a span of 360 feet, the slope being 30° with the horizon, and the base elevated 80 feet from the ground. The upper portion of the cone was cut away, leaving an opening 100 feet in diameter, over which another conical dome was erected on a drum 34 feet high, on the top of which was a third dome 24 feet diameter on a drum 28 feet high, making a total altitude from the ground to the summit of 280 feet, the whole being constructed of wrought iron. A full description of this dome will be found in the "Transactions of the Institute of Architects, for 1874."

CHAPTER IV.

BUILDING STONES.

29. The stones used for structural purposes in this country are divided into five distinct classes, namely: 1, Granites; 2, Sandstones; 3, Limestones; 4, Oolites; 5, Magnesian Limestones.

These stones are all a combination of several simple bodies such as silica, lime, magnesia and alumina, which are their chief ingredients; whilst iron, in the form of oxide, and potash are found in smaller proportions. *Silica*, or *silicic acid*, which forms a large part of many stones, is itself a compound of oxygen with the element silicon, and is found in its purest form in quartz, flint and sand. It is exceedingly hard so as to be able to scratch glass, and is unaffected by acids, but is acted on by alkalies when heated with them. *Lime* is a compound of oxygen with the metal calcium, but is never obtained pure, being mostly found as a carbonate, or in combination with carbonic acid, for which it has a great affinity, and which it will absorb from the atmosphere if left exposed for a length of time. The lime used in making mortar is obtained from the carbonate by driving off the carbonic acid and water in a kiln, and is then in the form of *quick-lime*, which absorbs water with great avidity and development of heat, and then becomes *slacked-lime*. Lime is also obtained in combination with

other acids, especially sulphuric acid, with which it forms the sulphate of lime, as in alabaster and gypsum, from which *Plaster of Paris* is obtained by heating and driving off the water, so that when water is added to the plaster it again becomes a hard and solid mass. Sulphate of lime is nearly insoluble in water, but the carbonate is readily dissolved. We can easily distinguish the two substances by the application of a little dilute acid, which produces violent effervescence in the carbonate, owing to its giving off carbonic acid, while the sulphate is unaffected. *Magnesia* is a compound of the metal magnesium with oxygen, and is generally obtained in the form of carbonate; this is not so readily acted on by dilute acid as the carbonate of lime, a very slight effervescence only being produced. Magnesia also occurs sometimes in combination with sulphuric and silicic acids. *Alumina* consists of the metal aluminium combined with oxygen, and is mostly found in combination with silica, forming the silicate of alumina which is the basis of the various clays.

Iron is never found in the pure metallic state on account of its affinity for oxygen, with which it forms the oxide or peroxide. It is seldom entirely absent from any mineral substances, the varied tints of which are chiefly due to its presence in greater or less proportion. The red and reddish-brown stones contain the largest quantities of iron, while those which are light brown, or cream-coloured, contain only a slight trace of it. *Potash*, which is the metal potassium combined with oxygen, is a powerful alkali, and is found in small quantities in granites and some other stones.

The above-named substances are said to be "chemically" united to the oxygen and acids with which they

combine, the combining proportions being always the same in the same materials, and the substance which results from the combination differs entirely from the original elements, the peculiar features of which have been lost or merged into those of the compound body. When, however, two or more of these compounds are found mixed together in a stone, they are said to be "mechanically" united, their proportions varying considerably, while the bodies themselves still retain most of the characteristics they possessed when separate. Thus we find silica combined *mechanically* with carbonate of lime in every variety of proportion, there being no *chemical* union between them.

30. Granites are classed by geologists among what are termed "metamorphic" rocks, having been originally deposited in a very different condition to that in which they are now found. They possess a more or less crystalline character, which has been produced by long-continued chemical action combined with great heat and pressure, by which their original nature has been entirely transformed. Granites differ considerably in the qualities of durability, hardness and crushing strength, some being readily disintegrated by the action of weather, while others possess great durability, and may be safely employed to resist the constant wearing effects of water. Their resistance also to crushing varies very greatly, specimens of Cornish granite having been crushed with a weight of 6,400 lbs. per square inch, while others from Peterhead and Aberdeen have required a pressure of 8,300 lbs. and 10,900 lbs. per square inch to crush them. The specific gravity of granite varies from 2·6 to 3; and the weight per cubic foot from 162 lbs. to 188 lbs.

The component parts of granite are quartz, feldspar,

mica or talc, and sometimes hornblende. These are *mechanically* united in various proportions, each of the three being a distinct mineral composed of bodies which are *chemically* united. *Quartz*, which forms about 70 per cent. of the composition of granites, is nearly pure chrystallized silica or silicic acid, and is the hardest of all the substances found in the composition of rocks; and the durability of stone often depends on the proportion of silica which it contains, it being perfectly insoluble in water or acids, although it is soluble in powerful alkalies. *Feldspar* is the mineral which gives the varieties of tint to granites, the colour being chiefly owing to the greater or less proportion they contain of the peroxide (rust) of iron. In 100 parts of feldspar, 65 are generally silica, 18 alumina, which is the basis of clays, and 16 potash. Those granites which contain a large proportion of feldspar, are not suited for external building purposes, as this material is the first component which is decomposed by the action of weather or of weak acids, and is converted thereby into a soft friable mass of earthy matter. *Mica* consists of thin transparent plates of great hardness, and to it is due the glittering appearance observable in granite. It contains 40 to 50 per cent. of silica, 16 to 37 per cent. of alumina, $7\frac{1}{2}$ to 10 per cent. of potash, and a considerable proportion of the peroxide of iron.

Hornblende is a heavy, dark-coloured crystalline substance found in a species of granite known by the name of "Syenite"; silica composes nearly one-half of its weight, lime and magnesia about one-third, and alumina one-eighth; it also contains iron and manganese in varying proportions. The presence of hornblende generally adds to the strength and durability of the granite.

Granite is quaried in various parts of England, Wales, and Scotland; the grey being found in Devon, Cornwall, and the Channel Islands; grey and pink granite at Mount Sorrel, in Leicestershire; a reddish kind in Cumberland and Westmoreland, as the Shap granite, which is remarkable for its handsome appearance when polished. In Scotland both red and grey are got from Peterhead, in Aberdeenshire, and a pink kind is found in the Isle of Mull, on the west coast, which is highly prized for ornamental purposes. It is also quarried extensively in the counties of Perth, Kinkardine, Banff, Inverness, Kirkudbright, and Wigtown. All the harder granites are susceptible of a high degree of polish, which greatly increases their beauty and durability. Granite lies at the bottom of the strata forming the crust of the earth, but from the action of subterranean forces it has been lifted up, so that it forms the substance of some of the highest mountains.

Serpentine is a stone that may be classed among granites on account of its *metamorphic* character, although it differs entirely from them in composition and structure, being chiefly composed of silica and magnesia in nearly equal proportions, and coloured with the oxide of iron. It is not suited for external use, but is valuable as an ornamental stone, being capable of receiving a good polish, which developes the beauty of its tints. It is chiefly obtained from the Lizard in Cornwall, and from Holyhead in Anglesea.

Slate is another substance found among the oldest rocks, which is metamorphic in character, its composition being chiefly silica and alumina. By the action of heat, combined with enormous pressure, its natural bed has generally been destroyed, and a new kind of bedding

called *cleavage* produced, so that it can be readily split into thin slices on the planes of cleavage. It is remarkable for its cohesive strength, as well as its power of resisting compression, both of which are from 8 to 10 tons per square inch in the best Welsh slates. Slate is obtained in several parts of North and South Wales, in Cumberland and Westmoreland, and the northern part of Lancashire. In Scotland slate is found in the counties of Argyle, Perth, Forfar, Aberdeen, and Banff.

31. SANDSTONES are so called from being chiefly composed of sandy or siliceous particles, which are cemented together by a small proportion of carbonate of lime. They all contain more or less of the peroxide of iron, to which material their variety of colour is owing; the red and brown sandstones containing as much as three per cent. of iron, while in the light-coloured stones only a slight trace is to be found. Sandstones also vary greatly in quality, some being soft and easily reduced to powder from want of a proper material to cement the particles of silica together; whilst others which have a good cement are among the hardest of the rocks. They may also be said to belong to nearly all the main divisions of the geological crust of the earth; those found in the lower or more ancient stata being harder, heavier, and more durable than those found in the higher or more recent strata. Thus, for example, in this country we have sandstones in the "Chalk" formation of Surrey, Sussex, Hants, and Dorset, which weigh only 103 lbs. to 111 lbs. per cubic foot, and these are generally of a soft and friable nature. Others are found in the "Wealden" which underlies the "Chalk," weighing 118 lbs. per cubic foot, as the Calverley stone, near Tonbridge Wells. In the "Lias," which is below the "Wealden," we

find sandstones weighing 127 lbs. per cubic foot, as in the neighbourhood of Whitby. In the "New Red Sandstone" series which underlies the "Lias," there are sandstones, such as that of Mansfield, weighing 146 lbs. to the cubic foot, and having a crushing strength of 5,000 lbs. per square inch. The Mansfield stone forms in its chemical composition a connecting link between the sandstones proper and the magnesian limestones, being found to contain 49 per cent. of silica, 26½ per cent. of carbonate of lime, and 16 per cent of carbonate of magnesia. The red Mansfield stone is considered to be less durable than the white.

There are many other quarries of serviceable stone in the "New Red" series, as in the neighbourhood of Newcastle and Whitehaven; Northallerton, Stockton, and Richmond in Yorkshire; in Lancashire, near Liverpool, Preston and Barrow; in several parts of Cheshire, Derbyshire, Staffordshire, Warwickshire, Salop, and Gloucestershire.

By far the largest number, as well as the most useful, of the sandstones are obtained from the "Carboniferous" series, which underlies the "New Red Sandstone;" for although this series derives its name from the presence of layers of coal, yet the larger proportion of it consists of sandstone rocks, varying greatly in texture and character, but known generally by geologists as the "grits." They are all more or less laminated, which enables us in most cases to detect readily the natural bed of the stone; some are so highly laminated that they split up with a slight blow into thin slabs or "flags," and are then only fit for paving purposes. The "grits" are generally very durable *when laid on their natural bed*, but if placed in front of a wall with their bed vertical they will flake off like the leaves of a

book, when exposed to the action of the atmosphere. Their weight varies from 140 lbs. to 160 lbs. per cubic foot, and the crushing strength from 4,000 to 8,000 lbs. per square inch. They contain from 93 to 98 per cent. of silica, with only a very small quantity of carbonate of lime; hence they are valuable stones for buildings which are exposed to a smoky or damp atmosphere, as weak acids do not injure them.

The chief quarries of the "grits" are found in Yorkshire, the term "York" being given in London to most of the stone which comes from that county. Some of these are in the form of "slabs" and "flags" for paving purposes, and are chiefly obtained from the vicinity of Halifax and Leeds, as well as in the parts of Lancashire bordering on Yorkshire. Other stones, such as those of Bramley-fall and Park-spring, near Leeds, and from Huddersfield and Halifax, are well suited for the best architectural work. The stone of Darley-dale, in Derbyshire, is a compact grit of brownish colour, possessing a high degree of crushing strength; and also the Craigleith stone, found near Edinburgh, which is less dense, but has a greater strength. The red sandstone of Dundee has a fine close grain, but is softer and less durable than many of the other stones belonging to this series.

The formation next below the "Carboniferous" in geological sequence, and known as the "Old Red Sandstone," contains many varieties of sandstones suitable for building purposes, some of which are highly durable, while others, which contain a considerable proportion of iron, are found to decompose rapidly. They are heavy stones, weighing from 150 lbs. to 160 lbs. per cubic foot. This formation is largely developed in several parts of Scotland, and some good building stones are obtained at

Dumfries, Corsehill on the Solway Frith, Roxburgh, Jedburgh, Berwick, Lanark, and many other places in the southern counties. In the central counties there is good stone to be had near Dundee, Forfar, and Stonehaven; and in the northern parts at Cromarty and Inverness. The "Old Red" is found on the West of England, and forms a large part of Salop, Hereford, and Gloucester; as at Lydney, Tortworth, Coleford, Newnham, Leominster, Malvern, and Chepstow. A similar formation is also found in Devonshire and Cornwall, where red and grey sandstones are obtained in large quantities. Red is the predominant tint in the stones of this formation.

32. Limestones are so called from being chiefly composed of the carbonate of lime, and are found more or less in nearly all the geological strata. They vary greatly in texture, some having the softness of chalk, and others a hardness equal to that of granite. Those which are not fit for use as building stones are valuable for burning into lime. The weight of the "freestones" which can be used for building is from 130 to 150 lbs. per cubic foot, and the crushing strength from 2,000 lbs. to 6,000 lbs. per square inch. The Chilmark stone (Wilts), belonging to the Oolitic series, has a crushing strength of 6,395 lbs. per square inch, and contains $10\frac{1}{2}$ per cent. of silica, 79 per cent. of carbonate of lime, 3·7 per cent. of carbonate of magnesia. It weighs 151 lbs. per cubic foot.

Some portions of the chalk formation produce limestones suitable for building purposes, as that of Totternhoe in Bedfordshire, of Beer near Axminster, of Devizes in Wilts, of Andover, Basingstoke, and Sherborne in Hants, and at many other places. The Kentish Rag is a hard limestone found in the lower "Greensand formation," and is much used for rough walling. In the "Wealden"

strata is found a fossiliferous limestone which goes by the name of "Sussex Marble," being sufficiently hard to receive a polish, and is found in the neighbourhood of Petworth and Staplehurst. In the "Lias" formation there are some good limestones at Shepton Mallet, Wellington, Yeovil in Somerset; at Merthyr in Glamorgan; at Ketton, in Rutland; at Whitby, in Yorkshire; and at Castle Carey, in Dorset. A very hard limestone is found below the Coal measures, and is occasionally worked for building, as in the neighbourhood of Leicester, and of Bristol, and also in some parts of Scotland; it constitutes the marble of Derbyshire. The Devonian formation produces a considerable quantity of good building limestone in South Devon, some of which makes an excellent marble.

33. Oolites, which form a large and very useful class of building stones, are limestones of a peculiar construction, being composed of small particles, like the *roe* of a fish, cemented together. They contain little or no silica, being almost entirely composed of carbonate of lime with a small proportion of the carbonate of magnesia. The best of those found in this country is from the Island of Portland, which contains 1·2 per cent of silica, 95 per cent of carbonate of lime, and 1·2 per cent of carbonate of magnesia. Its crushing strength is 3,900 lbs. per square inch, and its weight varies from 132 lbs. to 136 lbs. per cubic foot. Bath stone contains no silica, but has 94½ per cent. of carbonate of lime, and 2½ per cent. of carbonate of magnesia. The crushing strength of that from the Box quarries is 1,492 lbs. per square inch, and its weight is 123 lbs. per cubic foot. The Ancaster stone has a similar composition to the Bath, but is harder and heavier, its crushing strength being 2,345 lbs. per square inch, and its weight 139 lbs.

per cubic foot. The oolitic formation lies geologically between the Chalk and the New Red Sandstone, and is divided in four distinct series, the upper of which is the *Portland* oolite, below this the *Oxford* oolite, next comes the *Bath* oolite, and lastly the *Inferior* oolite. *Portland oolite* is so called from the island of that name on the Dorset coast, which is entirely composed of oolitic stone, but of variable quality, the quarries on the east of the island producing better stone than those on the west. In the same quarry also there is a great difference in quality, the stone of the middle or "Whitbed," generally known in London as *brown* Portland, being considered best for building purposes, although the lower bed is more often used on account of its whiteness and being easy to work. The stone of Chilmark and Tisbury, although not decidedly *oolitic*, belongs to the Portland series, which is also found in the counties of Bucks and Oxford. *Oxford oolite* is developed at Headington and Wheatly in Oxfordshire, the stone from which has been much used for buildings in Oxford; also about Weymouth and other parts of Dorset. *Bath oolite* is found in North Wilts, in the neighbourhood of Chippenham; also in parts of Gloucestershire, Northampton, Lincoln, and Rutland. There is much difference in Bath stone as regards durability and hardness, that which is called "Box ground" being generally considered best for outside work. *Inferior oolite* is found in Somerset, the stone of Doulting belonging to this series has been much employed for buildings of importance, and has a crushing strength of two tons per inch. It is also found at Hamhill near Yeovil, and Dundryhill near Bristol; at Painswick and other parts of Gloucestershire it yields an excellent building stone; the counties of Lincoln and Rutland also produce many quarries of the Inferior oolite.

34. MAGNESIAN LIMESTONE derives its name from being composed of the carbonates of lime and magnesia in nearly equal proportions. It belongs geologically to the New Red Sandstone series. The texture is very irregular, and consequently it is not to be always depended upon as a good *weather* stone; that which is most crystalline is generally the best for resisting the action of weather. The stone from Bolsover is the hardest and most compact found in this country, its crushing strength being 8,300 lbs. per square inch, and its weight 152 lbs. per cubic foot. It contains 3·6 per cent. of silica, 51 per cent. of carbonate of lime, and 40 per cent. of carbonate of magnesia; its specific gravity is 2·43. The weight of that from the Anston quarries is 144 lbs. per cubic foot. The name *Dolomite* is also applied by geologists to the formation producing the Magnesian limestone, which, however, occupies but a small tract of country in England on the edge of the coal-measures of Yorkshire, Durham, and Nottingham, extending from Mansfield in the south to Tynemouth in the north, and including Hartlepool, Darlington, Ripon, Tadcaster, Doncaster, and Mansfield.

35. POROSITY.—A very important feature to be considered in selecting stone for the outside walls of buildings is the amount of water which it will absorb when exposed to the action of driving rains. A large number of specimens of various kinds of stone, well dried, were left for several days in water, and being carefully weighed both before and after the saturation, the exact quantity of water absorbed was ascertained. Among sandstones, we find that specimens of Craigleith stone and from Park Spring (Yorkshire) absorb ·08 of their bulk of water; Kenton stone, ·099 of its bulk; Heddon stone, ·104 of its bulk;

time; the lime thus becomes the hydrate of lime and is said to be "slacked;" it then gradually "sets," and absorbs carbonic acid from the atmosphere, returning to its old condition of carbonate of lime, and, in conjunction with the sand, forms a siliceous limestone. Mortars that harden slowly should be protected from the weather for some time, as rainwater easily dissolves the hydrate of lime if exposed to its action before it has had time to become a carbonate.

The cohesive power of mortar is very slight while still moist, but as it dries this power increases, and in course of time may become as great as that of the stone or brick which it unites. Much, however, depends on the quality of the lime and the proportion of sand, about two or three of the latter to one of the former being generally used to make good mortar. *Fat* limes, or those which slack quickly when water is applied to them, do not generally make such strong mortar as *poor* limes, which take longer time in the process. It has been found that a great additional cohesive strength may be given to mortar by adding to it a small proportion of plaster of Paris, or, as it is called, rendering it "selenitic." The strength of mortar also depends much on the quality of the sand, which should be perfectly free from all earthy or clayey matters.

Some limes, when mixed with sand and water, have the property of setting or hardening in walls that are built under water; such limes are termed "hydraulic," and generally contain a considerable proportion of alumina, with silica and oxide of iron. The Lias formation which underlies the Oölites supplies the greater part of the stone for these limes.

38. Cements are artificial compounds of lime, iron,

silica or sand, and alumina or other materials: they do not require any admixture with sand, which only weakens them. When mixed with water they set rapidly, and become very hard in a few hours. The principal cements used for building walls are those known as "Roman" and "Portland," the former being a mixture of limestone with pozzolana, which contains all the ingredients mentioned above, besides small quantities of magnesia, potash, and soda. These are intimately mixed together in water, dried, and burnt in a kiln, and afterwards ground. Portland cement is generally a compound of limestone, such as chalk, with the mud obtained from river banks, the process of manufacture being similar to that of Roman. These cements are stronger after having been kept for some months in a dry place, than if used fresh. The cohesive strength of Portland, when used neat to form a joint between bricks or stones, is about 500 lbs per square inch, when thoroughly hardened. Cements for inside decorative purposes which go by the names of "Parian," and "marble," are composed chiefly of plaster of Paris mixed with borax or alum. They harden rapidly, and are capable of receiving a certain degree of polish.

39. Protection of the face of stones which are much exposed to weather or the action of acids is of great importance in this climate, and various processes have been devised for preventing disintegration. Ransome's method is to coat the stone with a thin film of silicate of lime, which is done by first washing the surface with a solution of silicate of soda, obtained by boiling flints in a solution of caustic soda. The surface is then played upon with a solution of the chloride of calcium, obtained by dissolving lime in muriatic acid; a chemical change immediately

takes place, the silicate of soda changing to silicate of lime, which is insoluble in water, and the chloride of calcium changing to chloride of sodium or common salt, which is very soluble in water, and is easily washed off.

Ransome's concrete stone is prepared in a similar manner. Finely sifted dry sand is mixed with a small proportion of pulverised stone or carbonate of lime, to each bushel of which mixture is added one gallon of the liquid silicate of soda, and the whole is well mixed in a mill. The plastic mass thus obtained is put into moulds and well rammed with wooden instruments so as to fill up all the interstices. When turned out of the moulds the blocks are played upon with a cold solution of the chloride of calcium, by which the mass is quickly solidified. It is then immersed in a boiling solution of the same material, and when removed from the bath is washed with water to remove the salt deposited on the surface. The crushing strength of this material varies according to the nature of the substances used in its composition.

Browning's solution for protecting the surface of stone can be applied like paint with an ordinary brush after the surface has been thoroughly cleansed. The composition of this varnish is as follows: 85½ per cent by weight of benzoline, or other similar spirit, 10 of a resinous gum such as "gum dammar," 2 of wax, 2 of sugar of lead, and ½ per cent. of corrosive sublimate.

For full information respecting the building stones of this country the reader is referred to the report of the Commissioners on Stone for Building the Houses of

Parliament, most of which is reprinted in Gwilt's "Encyclopædia." Also to the mineral statistics of the United Kingdom in the "Memoirs of the Geological Survey of Great Britain," edited by R. Hunt, in which there is a short notice of all the important quarries in the kingdom.

CHAPTER V.

TIMBER.

40. Growth.—The various timbers used for the purposes of building are derived from the class of trees which botanists denominate "Exogens" or outward growers, the new wood being added to the *outside* of that formed during the previous year. All trees found in cold climates and also the greater part of those in the tropics, belong to this class. The mode of development is as follows: the first year's growth of the stem consists of a central part called the *Pith*, which is surrounded by a woody stem, covered on the outside by the *Bark*. In the second year the inner part of the bark separates from the wood, and sap forms between the wood and the bark, the new sap-wood being connected with the pith by means of cross passages called medullary rays, through which the secretions pass from the outside to the centre. The root absorbs juices from the soil and conveys them to the woody fibres immediately surrounding the pith, by which they pass upwards and throughout all the branches to the leaves. By means of the leaves the superfluous water is given off, and the fluid entirely changes its nature. It now descends through a series of tubes in the inner part of the bark, and is deposited so as to form the new wood, bark, &c. The circulation

of the sap will continue in trees after the *outer* part of the bark has been removed. The *pith* connects the root with the leaf-buds, to which it conveys nourishment; it is at first green, and filled with fluid, but loses its colour as it dries up when the tree gets old; this is the first part that decays in the live tree, the *dead knots* found in wood being the dried up pith of the branches, and many trees which appear sound outside will, when cut up, be found partly decayed in the middle. The cellular mass of the stem is pressed into plates of various thickness by the wedges of wood formed within it, so that when a transverse section is made of the stem these plates appear as a number of lines radiating from the centre, and are called the medullary rays. The medullary sheath consists of spiral vessels surrounding the pith, projections of which pass through it into the medullary rays; by means of this sheath oxygen is conveyed to the leaves, being obtained by the decomposition of water or of carbonic acid. Surrounding this sheath is the *wood* proper, which consists of concentric layers formed by successive deposits year after year of the nutriment which descends from the leaves. In countries which have a winter and summer, each layer of wood is the produce of one year's growth; the secretions are found most abundant in the oldest layers, and when these become filled up they cease to perform any vital function, and form what is termed *heart wood*. The bark also consists of concentric layers, being increased by additions to its inner layers so as to allow for the gradual distension of the wood beneath; the outer bark does not increase but splits off, and a new one takes its place. The bark serves the double purpose of being a protection to the new wood, and also a filter through which the

descending juices pass. The part immediately under the bark is called the *sap-wood*, being that in which the fluid descends, and is deposited year by year; the quantity is greater in some kinds of trees than in others, the oak having about three times as much as the chesnut, and fir trees having about four times the quantity. This part of the tree contains the largest quantity of *albumen*, which is a material that readily decomposes. Growing trees contain a large quantity of water, and timber that has been newly cut is found to have from 20 to 45 per cent. of water, according to the character of the wood, the harder woods having less water than the softer. Before the wood can be used for building purposes, much of this water must be driven off, either by exposure to the air or by heating to about 130° Fahrenheit. The durability of timbers depends, in a considerable degree, on the quantity of *turpentine* they contain, as this is nearly insoluble in water, and prevents the decay of the wood. *Tannin* is also a very serviceable material in timber, giving great durability to such woods as contain the greatest quantity of it, as for example the oak.

41. Decay.—When timber has been employed in a building, it is sometimes found, after a few years, to be in a state of decay, so that the safety of the whole structure becomes endangered. This generally arises from the use of new timber insufficiently seasoned. When a tree is fresh cut down it contains a considerable amount of moisture, especially in the part near the outside called the *sap-wood*. Before, therefore, being used in a building it should be well seasoned, so as to get as much as possible of the sap out of it; if this is not done a certain amount of fermentation will take place in the albumen contained inside the timber, which causes the deposition of the

spores of fungi upon it, as these are constantly floating in the air unperceived, and only waiting for a situation suitable to their development in order to deposit themselves. If the timber is in a damp place without ventilation or a free current of air, the fungi develop very rapidly into minute fibres, covering the whole surface of the wood with a white film; these fibres penetrate into all parts of the wood, from which they suck the juices, and soon leave nothing but a soft, dry, powdery mass, which will fall to pieces when touched. A free current of air is hostile to the growth of the fungi, while a stagnant condition of the air, especially if the temperature is high, accelerates their growth. If sap-wood is used in a building, it is always the first to be affected by decay.

The prevention of decay in timber is a subject which has occupied the attention of many scientific persons, who have invented various processes of more or less utility. The most important point, however, is to get rid of all the superfluous water and sap, which can be done by soaking the wood for a few days in water, and then gradually drying in air, or by the quicker process of placing it in a chamber raised to a temperature of 130°; but timber must not be over-dried, otherwise it becomes brittle. If a solution of chloride of mercury or corrosive sublimate is injected into the pores of the wood, it combines with the albumen, and renders it insoluble, and thereby prevents its decomposition. Steeping wood in a fluid obtained from tar, and called *creasote*, is a very effective method of preventing decay, but can only be applied to rough timbers, as joists, sleepers, or rafters. Washing the surface with a solution of sulphate of copper or of corrosive sublimate, will prevent the formation of fungus on the surface

H 2

of wood, and the latter is effective against the attacks of insects.

42. Beams.—Timber, when used as a beam, may be loaded in several different ways; either fixed at one end and loaded at the other, or over its entire length; or supported at both ends and loaded in the middle, or loaded at several points, or loaded uniformly over its whole length. For convenience of calculation we shall give a distinct formula for each mode of loading. In obtaining these formulæ, it is assumed that the resistance of the material to extension and compression is exactly in proportion to the tensile and compressive strains applied; also that the neutral axis coincides with the centre of gravity of the section (**10**), the strains on the beam being less than sufficient to cause permanent injury to the fibres.

It has been previously shown (**12**), that when a beam is fixed or supported in a horizontal position and strained by a force (W), the "moment of resistance" of the beam at any section is directly as the *moment of inertia* of the section, taken about its *neutral axis*, and inversely as the distance of the neutral axis from the farthest edge of the section.

When a rectangular beam A B is fixed in a wall at A (fig. 16)* and strained by a weight W at the other end B, the *moment of the strain* M at any point D (where B D $= x$) has been shown (**8**) to be

$$M = W . x,$$

and this must be equal to the *moment of resistance*, which has also been shown to be

$$M = S \frac{I}{z},$$

* Page 21.

where I is the moment of inertia of the section, z the distance of the neutral axis from its furthest edge, S the strain per square inch of section, and depending on the nature of the material. The section of the beam being a rectangle of depth d and breadth b, we have by (12), $I = \frac{1}{12} b \,.\, d^3$, and $z = \frac{1}{2} d$.

Hence it follows that

$$W \,.\, x = \tfrac{1}{6} S \,.\, bd^2.$$

The strain is of course greatest when $x = l$, the length of the beam, or we have for the case of a beam fixed at one end and loaded at the other,

$$W = \tfrac{1}{6} S \frac{bd^2}{l} \quad . \quad . \quad (A).$$

When the weight W is uniformly distributed over the entire length of the beam, we have from (8) for the moment of strain at A,

$$M = \frac{W}{2} l = \tfrac{1}{6} S \,.\, bd^2,$$

$$\therefore W = \tfrac{1}{3} S \frac{bd^2}{l} \quad . \quad . \quad (B).$$

for a beam fixed at one end and uniformly loaded over its whole length. When the beam is supported at each end and loaded at the distance a from one end and b from the other, it has been shown (8) that $M = W \frac{a \,.\, b}{l}$, therefore in such a case

$$W = \tfrac{1}{6} S \frac{b \,.\, d^2}{a \,.\, b} l \quad . \quad . \quad (C).$$

If the load is in the middle, $a = b = \frac{1}{2} l$, and

$$W = \tfrac{2}{3} S \frac{bd^2}{l} \quad . \quad . \quad (D).$$

for a beam supported at each end and loaded in the centre.

When the beam is uniformly loaded throughout, we have, by (8), $M = \frac{1}{8} W l$, and therefore

$$W = \tfrac{4}{3} S \frac{bd^2}{l} \quad . \quad . \quad (E).$$

For a beam supported at each end, and loaded with $\frac{1}{2}$ W at each of two points dividing the length into three equal parts,

$$W = S \frac{bd^2}{l} \quad . \quad . \quad (F).$$

Comparing this with (D) we see that the beam will bear half as much more load than it will when W is at the centre.

The value of S has to be determined by experiment, for each kind of material. For if a beam, supported at each end, breaks with a load W at the middle, the value of S can be obtained from the equation,

$$S = \tfrac{3}{2} W \frac{l}{b \cdot d^2}.$$

Experiments by Barlow* on beams of the best English oak, 7 ft. long and 2 in. square, give W = 637 lbs. as the breaking weight in the middle. Substituting these values in the formula for S, we have,

$$S = \tfrac{3}{2} \times 637 \times \frac{84}{2 \times 4} = 10{,}033 \text{ lbs.}$$

In order to find the strength of any other beam of oak, we have only to substitute this value of S in either of the foregoing formulæ, (A), (B), (C), (D), (E), (F).

In a similar manner the value of S may be obtained

* Barlow, *On Strength of Material.* Lockwood & Co.

from the experiments of Barlow upon other kinds of timber, as given in the table below, the length being in feet, and all the other dimensions being put into inches in using the formulæ, and the breaking weight W in lbs.

	Value of S.
Red pine	671
Pitch ,, ...	816
Riga fir	540
Beech	778
Oak, British	836
,, Dantzic	730
,, Canadian	883

From the above formulæ it will be seen that the breaking weight at the weakest part of a rectangular beam is proportional to the breadth multiplied by the square of the depth, divided by the length. Also, by comparing (D) and (E), it appears that when the weight is equally distributed over the whole length of the beam, the breaking weight is twice as great as when concentrated at the centre. Hence we may consider the beam's own weight as straining it with a force equal to that produced by half its weight laid on the centre, and the beam may then be considered as without weight.

It is found by experiment that when the ends of a beam are firmly held down, the breaking weight is half as much more as when the ends are only supported loosely.

The permanent load laid upon a beam of timber ought not to exceed one-sixth of the breaking weight.

Example.—Let a beam, AB (fig. 19)* of English oak, having a breadth 8 in., depth 10 in., and length 10 ft., be loaded at D, 3 ft. 4 in. from A, and 6 ft. 8 in. from B; to find the breaking weight. By formula (C) we have,

$$W = \tfrac{1}{6}\,S\,\frac{bd^2}{ab}\,l = \frac{836}{6} \times \frac{8 \times 100}{3\frac{1}{3} \times 6\frac{2}{3}} \times 10 = 50{,}160 \text{ lbs.}$$

* Page 24.

If D is the centre of the beam, by formula (D) we have,

$$W = \tfrac{2}{3} S \frac{bd^2}{l} = 557 \times \frac{800}{10} = 44{,}560 \text{ lbs.}$$

Let the same beam be laid on its side; then we have $b = 10$ in., $d = 8$ in., and the breaking weight is,

$$W' = 557 \times \frac{640}{10} = 35{,}648 \text{ lbs.}$$

so that $W' : W :: 640 : 800$, or as $4 : 5$. This difference arises from the fact of the strength being proportional to the *square* of the depth, while it is only as the first power of the breadth. If, then, we double the breadth we only double the strength, but by doubling the depth we quadruple the strength.

If a prop or pillar is placed under the centre of a beam, supported at each end, it may then be considered to be divided into two separate beams, each of which being half the length will have double the strength of the unsupported beam, since the strength is inversely as the length; so that the whole beam will bear four times as much when supported in the middle as well as at the ends, as it will when only supported at the ends; and one-half the load will be borne by the pillar.

Suppose a beam AB (fig. 19, page 24), uniformly loaded, to be supported on three pillars at A, D and B; then the pillar at D will bear half the load of the beam, and the pressures on A and B will be inversely as their distances from D. If we call P the reaction at A, Q that at B, and R that at D, we find, by taking moments of the forces about A and B, that,

$$P : Q : R = DB : DA : AB,$$

$$\text{and } R = P + Q = \tfrac{1}{2} \text{ the load on the beam.}$$

Also the strain on the two parts of the beam, AD and BD, will be proportional to their length, and the load they will carry will be inversely as their length.

43. Safe-Load.—In following the several steps of the foregoing investigation, the student will notice a certain apparent contradiction between the hypothesis on which we start and the result we obtain. For whereas we begin by assuming that the strains are not such as to cause permanent injury, we end by applying the formula to the determination of the breaking-weight of the beam. As however in practice we seldom care to know the breaking-weight, but only the *safe-load*, it is better to ascertain what is the safe-load per unit of section that we can apply to the material of which a beam is composed, and determine therefrom the scantling that must be given to it, so that the strain upon any part shall not exceed that load. If we put f to represent the resistance of a square inch of the fibres at a distance of 1 inch from the neutral axis, then the resistance per square inch at top or bottom of the rectangular beam whose depth is d, is $f \times \frac{d}{2}$. Suppose the section of the beam to be divided horizontally into $2n$ equal rectangles, then the thickness of each is $\frac{d}{2n}$, and the *moment of resistance* of the several rectangles will be

$$2 f . b \left(\frac{d}{2n}\right)^3 \left(1^2 + 2^2 + \cdot\cdot\ n^2\right) = \text{M}$$

$$\text{or, M} = f . \ b \frac{d^3}{12}, \text{ nearly,}$$

when n is very great, or $\frac{d}{2n}$ very small.

And since the moment of W at the centre of a beam is

$\frac{1}{4}$ Wl, and must be equal to the above moment of resistance, we have

$$W = \frac{2}{3} f \frac{d}{2} \cdot \frac{bd^2}{l} \text{ .... (G)}$$

In order to be able to apply this formula to any particular beam we must ascertain the value to be given to f. Now the crushing strength of oak is 10,000 lbs per square inch, while its tensile strength is 15,000 lbs; but, as the strength of a beam is only that of its weakest part, we must take the lower amount as the breaking-strain per square inch at the top or bottom edge of the beam. Taking one-sixth of 10,000, or 1667, as the safe strain, we have

$$f \times \frac{d}{2} = 1667,$$

which gives for the *safe-load* in lbs at the middle of a beam of oak from formula (E),

$$W = 1111 \frac{bd^2}{l},$$

where all the dimensions are in inches.

Applying this to the example given above (**42**), where $b = 8$, $d = 10$, $l = 120$, we find the *safe-load* to be 7407 lbs., which is very nearly one-sixth of the breaking-weight as determined by the previous formula. If the beam is of fir, we must not take the safe strain per inch at more than two-thirds of what we have taken for oak; so that in fir we shall have $f \times \frac{d}{2} = 1111$, which gives from formula (G)

$$W = 741 \frac{bd^2}{l}$$

for the safe strain in lbs. at the middle of a beam of fir, all the dimensions being expressed in inches.

The crushing strength per square inch for different kinds of timber is given at page 113, so that by taking one-sixth of the value found in that table for the value of $f \times \frac{d}{2}$, we can determine the *safe-load* at the middle of any rectangular beam by means of the formula (G).

44. Deflexion.—A beam that is supported or fixed in a horizontal position, and subjected to a transverse strain, will be bent or deflected from the straight line, and will assume a curved form. The resistance to deflexion follows a very different law to the resistance to fracture. Let D be the amount of deflexion or distance of the centre of a loaded beam (supported at each end) from its position when unloaded; E the *modulus of elasticity* (**11**); W the load laid on at the centre; l the distance between the points of support; b the breadth and d the depth of the beam. Then it is shown by writers on the strength of materials, that when the *limit of elasticity* is not exceeded, the deflexion varies *directly* as the weight W and as the *cube* of the length l; it is also *inversely* as the *moment of inertia* (I) of the section. Hence we obtain the expression for the deflexion,

$$D = \frac{W}{48E} \times \frac{l^3}{I}.$$

And in a rectangular beam, $I = \frac{1}{12} b \,.\, d^3$ (**12**),

$$\therefore D = \frac{W}{4E} \times \frac{l^3}{b \,.\, d^3} \cdots\cdots\cdots \text{(H)}$$

The *resistance* to deflexion is, of course, *inversely* as the deflexion, and is expressed by the formula,

$$\frac{1}{D} = \frac{4E}{W} \times \frac{b \,.\, d^3}{l^3}.$$

That is to say, the *resistance to deflexion* is directly as the breadth into the cube of the depth, and inversely as the load into the cube of the length.

The value of E is found from the equation,

$$E = \frac{W}{4D} \times \frac{l^3}{b \cdot d^3}.$$

In the case of an oak beam 2 in. square and 84 in. long, supported at the ends and loaded with 200 lbs. in the middle, a deflexion of 1·28 was obtained. Substituting these values in the formula for E, we get,

$$E = \frac{200}{4 \times 1{\cdot}28} \times \frac{84^3}{2 \times 2^3} = 1447031 \text{ lbs.} = 646 \text{ tons.}$$

In a similar manner we obtain the values of E from experiments on other kinds of timber, as given in the following table:—

	Value of E in tons.
Red pine	821
Pitch ,, ...	547
Riga fir	574
Beech	604
Oak, British	646
,, Dantzic	520
,, Canadian	845

In using the formula (H) with these values of E, W must be expressed in tons. The formula for deflexion shows us the advantage gained by increasing the depth; for if we have two beams of equal length and breadth, but the depth of one beam double that of the other, the deflexions with the same load will be as 1 to 8, or by doubling the depth we increase the power of resisting flexure eight times.

When the load is uniformly distributed over the whole length of the beam, it can be demonstrated that the de-

flexion is reduced to five-eighths of what it is when the load is concentrated at the middle; so that the deflexion of a beam, arising from its own weight alone, is five-eighths of that which it would have if all its weight were to act at its centre. The deflexion of a beam, whose ends are firmly fixed, is half that of the same beam when the ends are only loosely supported.

Example 1.—To apply the formula (H) to ascertain the deflexion of the oak beam in (**42**) with W equal to 5 tons or one-fourth of the breaking weight laid on the middle. Here,

$$D = \frac{W}{4E} \times \frac{l^3}{b \,.\, d^3} = \frac{5}{4 \times 646} \times \frac{120^3}{8000} = \cdot 418 \text{ in.}$$

If the beam is laid on its side and loaded with 4 tons, or one-fourth of its breaking weight, we find $D = \cdot 522$ in.

Example 2.—Let it be required to find the breadth that must be given to an oak beam 10 in. deep and 10 ft. or 120 ins. bearing, in order that its deflexion D shall not exceed one-tenth of an inch when loaded in the middle with a weight of 1 ton. Here we have to calculate b from the equation,

$$b = \frac{W \,.\, l^3}{4E \,.\, D \,.\, d^3} = \frac{1728000}{258 \times 1000} = 6{\cdot}7 \text{ in.}$$

Now, suppose the breadth to be given, say 8 in., and other things being as before, it is required to find the depth of the beam. We have to calculate the value of d from the equation,

$$d^3 = \frac{W \,.\, l^3}{4E \,.\, D \,.\, b} = \frac{1728000}{258 \times 8} = 836.$$

$$\therefore d = \sqrt[3]{836} = 9{\cdot}42 \text{ in.}$$

45. LONG PILLARS.—When timber is subjected to a

compressing force in the direction of its length, as when used for columns, struts, &c., it will break either by bending, or by the crushing of its fibres, or by a combination of bending and crushing. This will depend on its length as compared with its diameter; for if its length is 30 times the diameter, the pillar will break by bending before crushing can begin. If its length is less than 25 times, but more than 10 times the diameter, it will break partly by crushing and partly by bending. If the length is less than 10 times the diameter, it will break by crushing only.

The mathematician Euler ascertained, from theoretical investigations on elastic rods, that the strength of long pillars in which the diameter is small as compared with the length, was proportional directly to the fourth power of the diameter, and inversely as the square of the length.

In the case of pillars of wood the experiments of Hodgkinson confirmed the rule obtained by Euler, and the formula for the breaking weight of pillars, whose length is 30 times the diameter, is

$$W = a \times \frac{d^4}{l^2}.$$

The value of a is 11 for Dantzic oak, and 8 for red deal, W being in tons, d in inches, and l in feet. In practice a pillar ought not to be loaded with more than one-tenth of its breaking weight; so that if we put 1·1 and ·8 for a, the above formula will give us the *safe permanent load* in each case.

In order to facilitate the calculation of this formula, we have given a Table of the second and fourth powers of numbers in the Appendix.

Example.—Let $l = 10$ ft., $d = 4$ in., $a = 1 \cdot 1$; then

$$W = 1 \cdot 1 \frac{4^4}{10^2} = 2 \cdot 8 \text{ tons, safe load.}$$

Mr. Hodgkinson found by experiment that if the load does not press on the pillar exactly in the direction of its axis, the strength is greatly diminished; and if it act in the direction of the diagonal, the strength is reduced to one-third of what it is when it acts down the axis.

46. Short Pillars.—When the length of a pillar is less than 25 times its diameter, but more than 10 times, the resistance to *crushing* comes into action as well as the resistance to *bending*, the material being partly crushed before it breaks by bending. Mr. Hodgkinson's method of approximating to the strength of such pillars is, first to calculate the breaking weight (b) by the formula for long pillars, as above; then if c is the area of the section of the pillar in inches multiplied by the crushing strength per square inch (as per table),* the breaking-weight (W) is found by the formula,

$$W = \frac{b \cdot c}{b + \frac{3}{4} c}.$$

Thus, in deal the crushing strength per square inch is about 3 tons, or between 6,000 lbs. to 7,000 lbs., and in oak 10,000 lbs., or $4\frac{1}{2}$ tons. Let us find, for example, the breaking-weight (W) of a pillar of oak, 4 in. square, whose length is 5 ft., or 15 times the diameter; then we have,

$$b = a \frac{d^4}{l^2} = 11 \times \frac{256}{25} = 112 \cdot 6 \text{ tons.}$$

$$c = 16 \times 4 \cdot 5 = 72 \text{ tons.}$$

* Page 113.

$$\therefore W = \frac{112 \cdot 6 \times 72}{112 \cdot 6 + 54} = 48 \cdot 7 \text{ tons.}$$

The *permanent safe load*, acting in the direction of the axis of the column, will therefore be 4·9 tons.

In pillars whose length is less than 10 diameters, the material is crushed before it can begin to bend, and therefore the crushing strength alone has to be considered, which is in proportion to the area of the transverse section; so that the strength of a short pillar is found by multiplying the number of square inches in the section by one of the numbers in the following table, which is the result of Hodgkinson's experiments on cylinders of wood, 1 in. diameter and 2 in. long, flat at the ends. The figures in the first column are for specimens moderately dry, while those in the second column are for specimens kept in a warm place for two months longer, after being turned. It was found that wet timber showed great weakness, being in some cases less than half as strong as dry timber. (See Mr. Hodgkinson's paper in the "Philosophical Transactions" 1840.)

In making use, however, of this table, it must be borne in mind that when a piece of timber is cut out of a tree, different parts of it will possess different degrees of strength. Thus the older wood near the centre will be much harder and stronger than that near the outside; the strength will also depend upon the age of the tree, older trees which have not begun to decay being generally tougher than younger ones. For these reasons the tabulated numbers can only be taken as giving an approximation to the relative strength of different kinds of wood, and are probably rather above the average strength.

Description of Wood.	Crushing strength per square inch in lbs.	
Alder	6,831	6,960
Ash	8,683	9,363
Baywood	7,518	7,518
Beech	7,733	9,363
Birch (American)	—	11,663
,, (English)	3,297	6,402
Cedar	5,674	5,863
Crab	6,499	7,148
Deal (Red)	5,748	6,586
,, (White)	6,781	7,293
Elder	7,451	9,973
Elm	—	10,331
Fir (spruce)	6,499	6,819
Hornbeam	4,533	7,289
Larch (fallen 2 months)	3,201	5,568
Mahogany	8,198	8,198
Oak (Quebec)	4,231	5,982
,, (English)	6,484	10,058
,, (Dantzic, very dry)	—	7,731
Pines, pitch	6,790	6,790
,, yellow, full of turpentine	5,375	5,445
,, red	5,395	7,518
Plum (wet)	3,654	—
,, (dry)	8,241	10,493
Poplar	3,107	5,124
Sycamore	7,082	—
Teak	—	12,101
Walnut	6,063	7,227
Willow	2,898	6,128

47. Tensile Strength.—The resistance of timber to a stretching-force acting in the direction of its length is proportional to the area of the transverse section, and quite independent of the length. The *ultimate strength*, or force that will break timber by stretching, is from $3\frac{1}{2}$ to 6 tons per square inch for fir, and from 4 to 8 tons for oak. Therefore to find the resistance of any piece of fir or oak to a stretching force, multiply the number of square inches in its section by one of these quantities.

The strain is here supposed to be applied in the direction of the fibres, but when the tensile force acts at right angles to their direction the resistance of the wood is very

much less, being about 1 ton per inch in oak, and $\frac{1}{3}$ of a ton in fir. For the tensile strength of other kinds of wood the reader is referred to "Carpentry and Joinery,"* in Weale's series, as edited by the present author.

48. Floors.—The strength of the timbers which carry a floor can be determined by the rules given above (**42**), (**43**), when we know the amount of load they are required to carry. In single-joisted floors the depth is generally 5, 7, 9 or 11 inches, so that if we suppose them to be placed 12 inches from centre to centre, and to bear safely a load of 100 lbs. to every square foot of flooring, we shall have a uniformly distributed load on each joist of 100 × L lbs. (L being its length in feet), which is equivalent to half that load at the centre. We can, therefore, in the case of fir joists, whose depth d and length l are given (in inches), find the breadth by means of the formula given above (**43**), namely,

$$b = \frac{W}{741} \cdot \frac{l}{d^2} = \frac{50L}{741} \cdot \frac{l}{d^2}$$

$$= \cdot 0675 \text{ L} \frac{l}{d^2}$$

The following Table of breadths is calculated from this formula, and gives the minimum scantling of single joists of different lengths required for safety when the load is 100 lbs. on every square foot of flooring.

L in ft.	$d = 5''$	$d = 7''$	$d = 9''$	$d = 11''$
	b in ins.	b in ins.	b in ins.	b in ins.
6	$1\frac{1}{4}$	...	...	...
8	$2\frac{1}{4}$	$1\frac{1}{4}$	...	...
10	$3\frac{1}{4}$	$1\frac{3}{4}$	$1\frac{1}{4}$	...
12	...	$2\frac{1}{2}$	$1\frac{1}{2}$	1
15	...	...	$2\frac{1}{4}$	$1\frac{1}{2}$
18	...	...	...	$2\frac{1}{4}$

* Lockwood & Co.

Where the ends of the joists are carried on BINDERS, the scantling of these must depend on the span of the joists as well as the length of the binder. Let S be the span in feet of the joists, L the length in feet of the binder, l and d its length and depth in inches, then for a given value of l and d we get—

$$b = \cdot 0675 \text{ L . S } \frac{l}{d^2}$$

The following table gives the value of b when the depth is 10 inches and 12 inches for different values of L and S; no allowance, however, is here made for notches or mortice holes cut in the binder for the ends of the bridging joists; where these are made, an extra thickness of one inch should be given on each side in which they are cut, as they considerably weaken a beam.

L in ft.	S in ft.	$d = 10''$	$d = 12''$
		b in ins.	b in ins.
10	6	3	...
,,	8	$5\frac{1}{4}$	$3\frac{3}{4}$
,,	10	$8\frac{1}{4}$	$5\frac{3}{4}$
12	6	$7\frac{1}{2}$	5
,,	8	$9\frac{3}{4}$	$6\frac{1}{2}$
,,	10	$10\frac{3}{4}$	$8\frac{1}{4}$
15	6	11	$7\frac{3}{4}$
,,	8	...	$10\frac{1}{4}$
,,	10	...	$12\frac{3}{4}$

When the binders themselves have their ends framed into GIRDERS, we have the case of a beam loaded at one or two intermediate points. Thus in a girder 10 or 12 ft. long there will be a binder framed into it at the centre, and in one of 15 or 20 ft. there will be two binders dividing the girder into three equal bays. In the former case we have half the weight W of the floor acting on

the middle of the beam, and in the latter we have the case provided for by the formula (F) (**42**), the strain being two-thirds of that produced when all the load is at the centre. When there is only one binder at the centre of the girder, if we put S for the span of the bridging joists, L the length of the binder, we have W = 100 S × L as the load on the centre of the girder, since half the weight of the floor is carried by the walls on which the ends of the bridging joists rest; so that

$$b = \cdot 135\ \mathrm{L}\ .\ \mathrm{S}\,\frac{l}{d^2}$$

gives the breadth of a fir girder whose length and depth are known, which will safely bear a load of 100 lbs. per square foot of flooring. As an example, let S = 6, L = 12, $l = 144$, $d = 12$; then

$$b = \cdot 135 \times 6 \times 12\,\frac{144}{144} = 9\tfrac{3}{4} \text{ inches.}$$

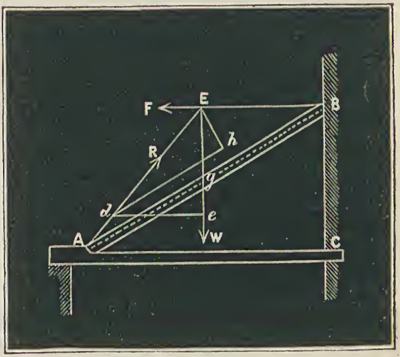
Let there be two binders 6 ft. apart framed into a girder 18 ft. long, the depth being 12 inches as before. In this case one-third of the load is carried by the walls, and the total weight sustained at two points by the beam is 100 × 2S × L, which by formula (F) is equivalent to a load of $\frac{2}{3}$ × 100 × 2S × L placed at the centre; or, $W' = \frac{4}{3}$ W, so that b will be $\frac{4}{3}$ of the breadth found in the former case; that is, b = 13 inches. An extra breadth should be given to allow for the mortice holes cut in the girder for the ends of the binders, but as these only occur in one or two places they do not materially injure the strength.

49. Timber Framing.—When a piece of timber is used in framework, it is subjected to one or more of the strains which we have been considering, and its proportions must

be arranged according to the amount, character, and direction of the strains it has to bear. When a beam is subjected to a strain in any direction, the straining force can be resolved into two other forces at right angles to each other, one acting transversely and another acting longitudinally, by means of the principle of the resolution of forces (2).

Suppose a rafter AB (fig. 39) carrying a load W acting at its centre of gravity g, to lean against a wall at B, and to be framed into a horizontal beam at A. Let F be the reaction of the wall acting horizontally at B; then R the reaction at A is the resultant of the two forces W and F, and its direction meets that of W and F in E. If then Ee is taken to represent on scale the weight W, the horizontal line de will represent F, and Ed will represent R, in direction and magnitude; or the three forces W, F and R are in the ratio of the three sides of the triangle Eed.

Fig. 39.



We can also determine the relation of F to W by equating the moments (4) of these forces taken about the point A, namely,

$$F \times BC = W \times \tfrac{1}{2} AC,$$

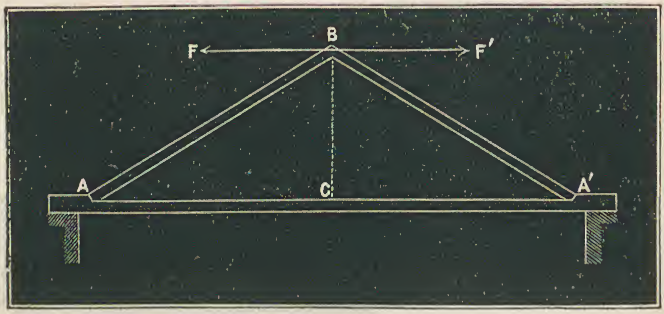
or,

$$F = W \frac{AC}{2BC}$$

Now, suppose the wall BC removed, and that in its place is substituted another rafter BA′ equal to AB, and

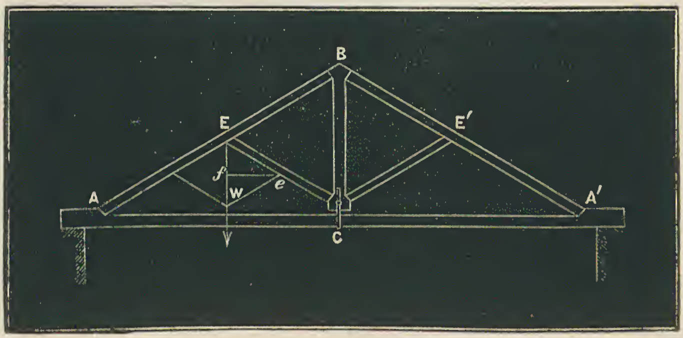
framed into the tie-beam AA′ (fig. 40); F will now be the mutual pressure of the two rafters at B, and the horizontal strain on the tie-beam will be represented by the line *de* (fig. 39); E*e* being the line which represents the

Fig. 40.



weight laid on each rafter. The tie-beam must also be made sufficiently strong to bear its own weight without bending, which it may also be prevented from doing by being held up in the middle by a piece of timber called a

Fig. 41.



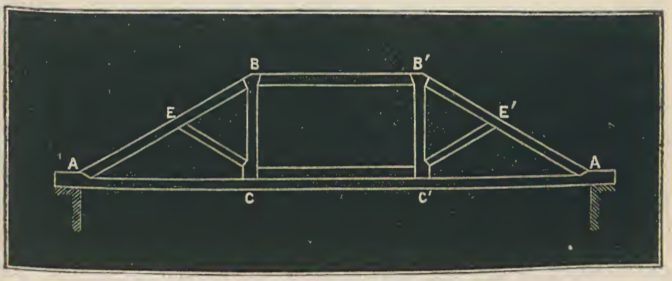
king-post, into the head of which the two rafters are framed (fig. 41); by this means the transverse strain on the tie-beam is reduced to one-fourth of what it is when

the king-post is absent, so that its dimensions may be reduced accordingly; the vertical strain on the king-post is *half* the weight of the tie-beam, and its scantling may be determined from the rule given for tensile strength (**47**), allowing about 1 ton per square inch of section as the safe strain.

The next step is to strengthen the rafters by means of struts CE and CE′ framed into the foot of the king-post, and supporting the centre of each rafter. The pressure at E of the load on the rafter may be considered as *half* the entire load laid on the rafter. Let EW represent the vertical load at E; draw W*e* parallel to AB and *ef* horizontal. Then E*e* represents the compression on the strut, which can be resolved into its vertical and horizontal components E*f* and *ef*, the former of which represents the *additional* vertical strain produced by *each* strut on the king-post; and the latter is counterbalanced by the horizontal thrust arising from the opposite strut CE′.

The tie-beam may be sustained in two places by means of queen-posts BC, B′C′ (fig. 42), into the heads of which

Fig. 42.



the rafters are framed, and their reaction transmitted by the straining-piece BB′, which is *compressed* by the force F, represented by *de* (fig. 39). Each queen-post sustains

one-fourth the weight of the tie-beam, by means of an iron strap passed round the tie-beam and bolted to the foot of the queen-post; and the tie-beam will be able to bear nine times as great a transverse strain as it would if supported only at the ends. Each queen-post has also to sustain the vertical component of the thrust from *one* of the struts, the horizontal components being counter-balanced by means of a straining-piece inserted between the feet of the queen-posts.

To find the strains upon the rafter AB (fig. 39), resolve the resultant R into its two components perpendicular and parallel to AB; these will be represented by the lines Eh and dh; then Eh represents the transverse strain at the centre, and dh the longitudinal compression. It will be seen from the figure that the horizontal thrust is greater for a low pitch than for a high one; putting R for the force Ed, acting down EA, and F for the horizontal thrust ed, we find when the angle B A C is 26° that F = W, and R = 1·4W; for B A C = 30°, F = ·87W, and R = 1·3W; for 45° pitch, we find F = $\frac{1}{2}$W, R = 1·12W; when B A C = 60°, F = ·3W, R = 1·04W. With a pitch of 26° we also find the transverse strain eh on the rafter to be ·444W, and the compression hd down the rafter is 1·33W; for a pitch of 30° these are ·423W and 1·23W respectively; for a pitch of 45°, they are ·355W and 1·052W; and for 60° they are ·25W and 1·01W.

The various strains to which the different timbers in a truss are subjected may also be determined graphically by Maxwell's diagram of strains (3), and an example of its application to a king-post roof will be found worked out in detail in the author's treatise on Carpentry and Joinery in Weale's Series* (page 108).

* Lockwood & Co.

50. Scantlings.—In order to be able to determine the scantlings that must be given to the timbers of a roof, we must know what weight it will have to carry. Tredgold gives the weight of a slated roof with the timbers as 26 lbs. per square foot of surface, to which he adds 40 lbs. per foot for the action of "wind and other occasional forces;" this is probably sufficient to allow for the pressure of the wind on a roof of moderate pitch; but as recent experiments have shown that the force of a high wind sometimes amounts to 50 lbs. per square foot on a plane surface exposed to its direct action, we must add as much as this where the roof is of high pitch. For low-pitched roofs we may therefore consider 66 lbs. per foot as the total pressure to be sustained, and in roofs of high pitch 76 lbs. per foot must be taken as the load; and the total load W sustained by the rafter AB of a trussed roof will be found by multiplying its length by the distance apart of the trusses, and the product by 66 or 76 according to the pitch of the roof.

The following rules are given by Tredgold for determining the scantlings of the several timbers of a trussed roof of *fir*. Let b be the breadth in inches, d the depth in inches, and l the length in feet of any beam:—

1. King-post, $b \times d = \cdot 12\, l \times s =$ area of section, where s is the span in feet.

2. Queen-post, $b \times d = \cdot 27\, l \times s =$ area of section, where s is the length of the part of the tie-beam which each post sustains in feet.

3. Tie-beam; s being the length in feet of the longest unsupported part; $d = 1\cdot 47\ \frac{s}{\sqrt[3]{b}}$, in inches.

4. Principal rafters having a king-post; $d = \cdot 96\ \times$

$\frac{l^2 \times s}{b^3}$, in inches, s being the span in feet. When there are two queens, the multiplier is ·155 instead of ·96.

5. Straining-beam between heads of queen-posts, where $d : b = 10 : 7$, or $b = \cdot 7\, d$; then $d = \cdot 9 \sqrt{l \sqrt{s}}$; s being the span in feet.

6. Struts and braces supporting rafters, of which s is the length unsupported in feet; where $d : b = 10 : 6$, or $b = \cdot 6\, d$; then $d = \cdot 8 \sqrt{b \sqrt{s}}$.

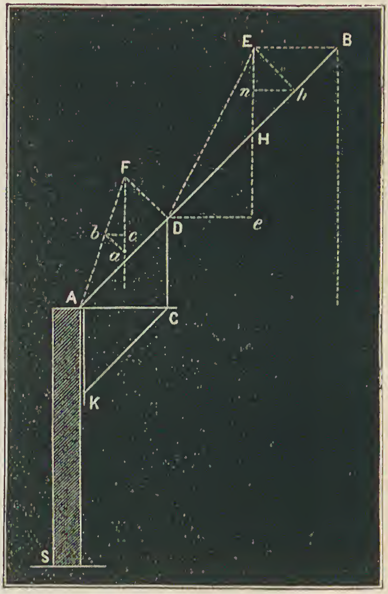
7. Purlins; s being their distance apart in feet, where $b = \cdot 6\, d$; then $d = \sqrt[4]{s . l^3}$.

8. Common rafters, where l is the bearing in feet, $d = \cdot 72 \frac{l}{\sqrt[3]{b}}$.

51. Hammer-beam Roof.—In this kind of roof-truss the tie-beam is omitted, and the portion of the rafter which rests on the wall is made rigid by being framed into four pieces of timber, as A D C K (fig. 43); a horizontal *hammer-beam* A C rests at one end on the wall and has the foot of the rafter framed into it as into a tie-beam; the outer end C supports a vertical *strut* C D, which is framed into the rafter at D; and the end C is also supported by a bracket or strut C K framed into a vertical wall-piece A K. We have, therefore, only to consider the thrust arising from the load in the portion of the rafter D B, the weight W of which may be supposed to act at H, and we determine the relation of the forces as before (fig. 39), by drawing the horizontal line B E to meet the vertical through H, and if Ee represents the weight W, the line E D will represent the pressure at D, and De the horizontal thrust F acting at D. By

means of the rigid piece of framing D K this thrust is brought down to the point K, and the lower this point is down the wall the less will be the tendency of the thrust to overturn it. The force F at K has a moment F × K S about the outer edge S of the wall, which is counteracted by the moments of the weight of the half truss and of the wall itself. If W is the weight of the upper part of the roof above D, w that of the frame D K, and R the weight of the wall itself, we may consider all these forces as acting down the middle of the wall; and in order that the structure may have stability we must have

Fig. 43.



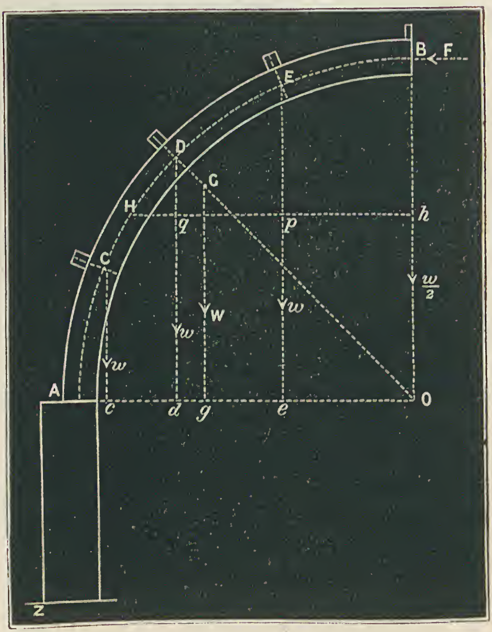
$$(W + w + Q)\frac{t}{2} > F \times K S.$$

The strains in the pieces A C, C K, may be determined as previously shown (2) in the case of a bracket. To find the compression down the strut D C, draw Eh perpendicular to B D, and hn horizontal, then En will represent the vertical pressure at D from the rafter D B. To find the pressure arising from A D, draw D F perpendicular to A D, meeting the vertical through the middle point of

A D at F; draw F A; let Fa represent the weight of the rafter A D, draw ab parallel to D F, and ab will be the reaction at D of A D; draw bc horizontal, and ac will represent the pressure down D C arising from A D; so that En + ac will be the compression in D C. The line Fc is the vertical pressure arising from A D upon the wall at A. In this manner the strains upon any part of the framing can be compared with the weight of the load upon the rafter.

52. Arched Roof.—Ribs or rafters of a semi-circular

Fig. 44.



form are constructed of planks bolted together side by side, three or more thicknesses being used in their con-

struction. Let A B (fig. 44) represent the half of such a rib, carrying the load of a roof by means of purlins at C, D, E, and B, and resting on a pier at A. Let W be the weight of the rib itself, acting at G, its centre of gravity, O G being ·9 O D by the table in the section on "centre of gravity" (6), the line B E D C passing through the centre of the rib; let w be the load on each purlin, acting at B, E, D, and C. At B only $\frac{1}{2} w$ must be taken. Then if F is the horizontal pressure, at B of the other half rib, we have to take the moment of F about A as equal to the sum of the moments of the weights; therefore the equation

$$\text{F} \times \text{B O} = \text{W} \times \text{A}g + w\,(\tfrac{1}{2}\,\text{O A} + \text{A}e + \text{A}d + \text{A}c)$$

determines the horizontal thrust F, which we can now consider as acting at the base A of the rib, and tending to overturn the pier about its outer edge Z. To resist this thrust we have Q the weight of the pier and W + $\frac{7}{2} w$, the weight of the roof acting down the centre of the pier; so that its thickness t that will suffice to produce equilibrium may be found from the equation,

$$(\text{Q} + \text{W} + \tfrac{7}{2}\,w)\,\frac{t}{2} = \text{F} \times \text{A Z}$$

If the pier is thicker than the dimension found by this equation, the structure will be in a condition of stability.

In order to determine the strength of the rib itself, we take moments of the loads at D, E, and B about H, where the strain is greatest, H being the middle point between C and D; and equate the sum of these moments with the *moment of resistance* of the section of the rib, namely,

$$w\,(\tfrac{1}{2}\,\text{H}h + \text{H}p + \text{H}q) = \text{S}\,\frac{bd^2}{6}$$

When fir is used for the rib we have (**42**), $\frac{1}{6}S = 15$ for safe-load; b and d being in inches, and the other dimensions in feet; w being in lbs. As an example, let the rib have an internal radius of 10 ft., and an external one of 11 ft., its depth and breadth being 12 in.; let W = 500 lbs., $w = 2{,}000$ lbs.; then we find $Ag = 4{\cdot}3$, $Ad = 3{\cdot}6$, $Ac = 1{\cdot}3$, $Ae = 7$, $OB = 10{\cdot}5$, $OA = 11$. Therefore the first equation becomes,

$$F \times 10{\cdot}5 = 500 \times 4{\cdot}3 + 2000\,(5{\cdot}5 + 7 + 3{\cdot}6 + 1{\cdot}3)$$
$$\text{or } F = 3520 \text{ lbs.}$$

Let $AZ = 10$ feet, and the weight of a cubic foot of the pier be 120 lbs. Then if Q is the weight of a pier 1 ft. broad, we have $Q = 1200\,t$; and the second equation becomes,

$$1200\,\frac{t^2}{2} + 7500\,\frac{t}{2} = 35200$$
$$\text{or } 12t^2 + 75t - 704 = 0$$
$$\therefore t = 5{\cdot}2 \text{ feet.}$$

In order to obtain stability the thickness of the pier must exceed 5·2 feet, when the length is 1 foot. If we suppose the ribs to be 10 feet apart, and each to be supported by a continuous wall 10 feet long, the equation becomes,

$$120\,t^2 + 75t - 704 = 0,$$

which gives $t = 2{\cdot}125$ feet for equilibrium; the quantity of walling, however, is in this case 4 times as great as in the former. For the strength of the rib at the point H, we find the *moment of strain*, when $w = 2{,}000$ lbs., to be 20,250, while the *moment of resistance* of the rib from the expression $M = 15\,bd^2$, is 25,920; so that in this

case the rib is stronger than necessary in about the ratio of 13 : 10. If we require to know the amount w that can safely be laid on such a rib, we obtain it from the equation

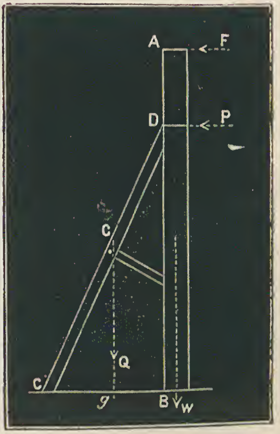
$$w = \frac{25920}{10 \cdot 125} = 2{,}530 \text{ lbs.}$$

To determine b when $w = 2{,}000$ lbs. and $d = 12$ in., we have,

$$b = \frac{2000 \times 10 \cdot 125}{15 \times 144} = 9\tfrac{1}{2} \text{ in.}$$

53. Shoring.—The object of a *shore*, when fixed obliquely against a vertical wall, as DC against the wall AB (fig. 45), is to prevent its being overturned by the pressure of some horizontal force F acting at the summit. The *raking-shore* DC supports the wall by means of a piece of timber called a *needle* driven through it at D, against which the shore is firmly fixed by wedges at its base C, so that it cannot be overturned without lifting up the portion of the wall above D. If W is the weight of the wall acting vertically through its centre of gravity, t its thickness supposed to be uniform, H its height; when the wall is about to turn over about its base B without the shore, we have $F \times H = W \times \frac{t}{2}$.

Fig. 45.



The shore being fixed firmly at D at a height h above B, it produces a horizontal pressure P against the wall to counteract that of F; so that

$$P \times h = F \times H = W \frac{t}{2},$$

$$\text{or, } P = \tfrac{1}{2} W \frac{t}{h}.$$

The shore must be prevented from turning over about its base C by the weight w of the part of the wall AD, together with its own weight Q acting at its centre G. We can therefore determine what weight w will suffice to produce equilibrium by taking moments of these forces about C; so that

$$w\left(\frac{t}{2} + BC\right) + Q \times Cg = P \,.\, h = W \frac{t}{2},$$

$$\text{or, } w = \frac{W \,.\, t}{t + 2BC}$$

For example, let $H = 30$ ft., $t = 1{\cdot}5$ ft., $BC = 5$ ft., and suppose the wall to have a frontage of 12 ft., and to weigh 1 cwt. per cubic foot; then $W = 30 \times 1{\cdot}5 \times 12 = 540$ cwts.: let $Q = 4$ cwts., then we find $w = 70$ cwts., so that AD is 4 ft., which is the minimum amount of walling above D that will suffice to prevent the force F from overturning it.

The compression down the shore is the resolved part of the forces w and P, which is

$$w \frac{BD}{CD} + P \frac{BC}{CD}$$

or, substituting for P its value, $\frac{1}{2} W \frac{t}{BD}$, we have

Compression down the shore $= \frac{1}{CD}\left(w \times BD + \frac{W.t.BC}{2BD}\right)$

Apply this to the above example, where BD = 26, BC = 5, $t = 1{\cdot}5$, and CD = 26·5, $w = 70$, W = 540, CD = 26·5; then we find the compression down the shore to be = 71·6 cwts. To find the necessary strength of the shore we use the formula for long pillars (**45**), where the safe-load in cwts. for a pillar of fir is

$$\cdot 8 \times 20 \frac{d^4}{l^2},$$

from which we can get the value of d when $l = 26{\cdot}5$, and the load is 71·6, namely, $d = \sqrt[4]{\frac{71{\cdot}6}{16} \times 702} = 7{\cdot}5$ ins., supposing the shore to be die-square.

There is also a strain on the shore at right angles to its length, which is equivalent to a load at the centre of a horizontal beam, namely

$$\frac{1}{2CD}\left\{\tfrac{1}{2}\,W\,.\,t + (Q + w)\,BC\right\}$$

which, in the foregoing example, amounts to 14·6 cwts., or 1635 lbs. Now, the safe load in a beam of fir is (**43**)

$$62\,\frac{bd^2}{l}$$

in lbs., where l is in feet, b and d in inches. In this case we have to determine b when $d = 7{\cdot}5$ ins., $l = 26{\cdot}5$ ft., from the equation

$$1635 = 62\,\frac{bd^2}{l}$$

$$\text{or, } b = \frac{1635}{62} \; \frac{l}{d^2} = 12{\cdot}4 \text{ inches.}$$

If the breadth $b = 7{\cdot}5$, then d will be found from the same formula to be 9·6 inches.

If a strut is fixed at G it will prevent the bending of the shore, and will increase its transverse strength four-fold. The compression down the strut will be the load at the centre of the shore, or, in this case, 14·6 cwts.

54. Centerings for arches are structures of timber, temporarily erected for the purpose of sustaining the voussoirs until the arch is completed and is able to sustain itself. On account of the friction of their surfaces, the voussoirs do not begin to press upon the centering until the joints make an angle of 30° with the horizontal; the pressure then increases slowly up to 45°, when it amounts to one-fourth the entire weight of the voussoir. After the angle of 45° is passed the pressure of each stone increases more rapidly, being about one-third the weight of the voussoir at 50°, and at 60° rather more than half the weight of the voussoir. After this the stones soon press with their whole weight on the centering; and any voussoir in which a vertical line dropt from its centre of gravity, falls outside its lower joint, may be considered as pressing with its full weight on the centering. This variation in the pressures of the stones upon the centering is the chief matter to be considered in designing a centre, as greater strength and support must be given to the parts near the crown than to those at the haunches. The laying of the voussoirs must also be proceeded with uniformly on both sides of the arch after the angle of 30° is passed, so as to prevent the frame-work from being distorted by unequal pressure on the two sides.

When the depth of a voussoir is double its thickness, it will rest its whole weight on the centre at an angle of 60°; if less than double, then its full weight will come on the centre at a lower angle; and if greater, at a higher angle than 60°; so that with short voussoirs the pressure on the centering is greater than with long ones.

CHAPTER VI.

IRON.

55. IRON is a metal that is always found in nature combined with other substances, such as oxygen, carbon, phosphorus, sulphur, silica, alumina, and other earthy matters, from which it has to be separated by exposing the ore, or *ironstone*, to a very high degree of temperature. The ironstone is first roasted by heating with small coal to drive off its moisture and render it more easy of reduction to the metallic state in the smelting furnace, where it is heated by means of a strong blast with limestone and coal, the former of which combines with the silica and alumina, while the oxygen of the iron-oxide unites with the carbon of the fuel, leaving the molten metal at the bottom of the furnace, which is run off into blocks called "pigs," containing from 88 to 96 per cent. of pure iron, combined with from $2\frac{1}{2}$ to 5 per cent. of carbon, and other matters as sulphur, phosphorus, manganese and silica, in smaller proportions. The carbon adds considerably to the strength of the iron, if in moderate proportion, but the other substances are a source of weakness, and should be got rid of as much as possible. Iron is employed for structural purposes, under three different forms, namely:—(1) Cast-iron; (2) Wrought-iron; (3) Steel.

56. CAST-IRON is the name applied to the metal that has been run into moulds when in a liquid state, and is

made from the "pig" by heating with coke in a cupola furnace, where a portion of the carbon and other impurities are removed, and the metal is poured into moulds made of a peculiar kind of sand mixed with charred oak in fine powder. The temperature of the molten iron is about 2,700° Fahr. Cast-iron is brittle, and readily broken by a sudden blow, the fracture generally presenting a crystalline appearance; its brittleness is increased by the addition of carbon. It also flies into fragments if cold water is applied to it when in a heated state; hence its use must be avoided in buildings intended to resist the action of fire, or where it would be subjected to sudden jars or changes of load.

The ultimate crushing strength of cast-iron being six times as great as the resistance to stretching, it becomes a valuable material for columns which have to sustain a vertical pressure.

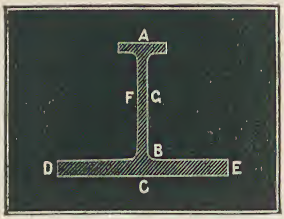
When beams are formed of cast-iron they require to be carefully tested before being fixed, as this material is liable to flaws from carelessness in the casting, or from being allowed to cool too rapidly, whereby it becomes honeycombed in the middle.

The material known as "malleable cast-iron" may be made by covering up a casting with clay, heating to redness and then allowing it to cool slowly, whereby the metal loses much of its brittleness, and becomes annealed. Another method is to heat it with such substances as powdered charcoal, bone-ash, forge scales, red iron-oxide, or manganese oxide, by which much of the carbon is oxidized and removed from combination with the metal.

57. Cast-iron Beam.—The strongest form of cast-iron beam of given *weight* and *depth* was determined by Hodgkinson, after numerous experiments, to be that in

which the top and bottom flanges are to one another in the inverse proportion of the resistance to stretching to the resistance to crushing, or as six to one; since the top flange is subjected to compression while the bottom is subjected to extension. The section of th ebeam in the centre is that of (fig. 46), two-thirds of the metal being contained in the lower flange.

Fig. 46.



An approximate rule for the strength of such a beam is,

$W = 2{\cdot}2 \dfrac{A\,d}{l}$ = breaking-weight in tons at the centre;

where A is the area in square inches of the section of bottom flange in the middle, d to the total depth A C of the beam in inches, l the span in feet. This rule is formed on the supposition that the strength of the flanges is so great that the resistance of the middle part is small in comparison, and may be neglected.* As an example, let $A = 4{\cdot}4$, $d = 5\frac{1}{8} = \frac{41}{8}$, $l = 54$; then

$$W = 2{\cdot}2 \frac{4{\cdot}4 \times 41}{8 \times 54} = 10{\cdot}86 \text{ tons.}$$

The permanent load on a cast-iron beam should never exceed one-sixth of the breaking weight. When a cast-iron beam is heated to redness it is found to have only two-thirds of the strength that it has when cool.

Another approximate rule for the strength of such a beam is given by Hodgkinson, in which the effect of the vertical part or web is taken into consideration, and the

* Hodgkinson, *On Cast Iron.*

operation of the top flange of a beam, when strained, is supposed to be that of a fulcrum on which to break the bottom flange and web. Putting d for the total depth AC, d_1 for the depth AB to the top of the lower flange, b the breadth DE of lower flange, b' the thickness FG of the web (all in inches), and l being the length of bearing in feet, the breaking-weight in the centre, in tons, is

$$W = \frac{2}{3\,d\,.\,l}\left\{b\,.\,d^3 - (b - b')\,d_1^3\right\}.$$

This formula depends on two suppositions: 1st, that all the particles, except those in the top flange of a bent beam, are in a state of tension; 2nd, that the resistance of each particle is proportional to its distance from the top of the beam.

When the depth of a cast-iron beam is uniform throughout, it would be a great waste of metal to have the lower flange of the same width at the ends as at the middle, and in order to obtain a beam of uniform strength at every part, the plan of the lower flange should be in the form of a double parabola, or flat segment of a circle (fig. 47); for the strength varies as the breadth $d\,e$ at any part, and also the strain is, by (8), as the product of Ae × Be, and in the parabola de varies as Ae × Be. If, on the other hand, the lower flange is kept of the same width throughout, a beam of uniform strength may be

Fig. 47.

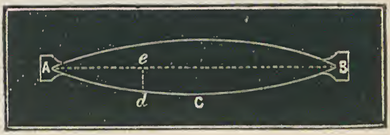
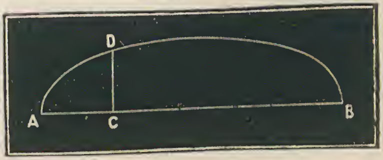


Fig. 48.



obtained by making the depth vary, so as to form an ellipse (fig. 48); for the strength at any part is as the square of D C, and also the strain is as A C $\times$ B C, which expresses the relation between the co-ordinates of the ellipse. [Methods of drawing these curves will be found in the author's Practical Geometry.]

The *safe load* on a beam of this form can be calculated for any given strain per square inch in the manner previously employed for beams of wood (**43**), the centre of gravity G (fig. 11, page 15) being determined by the process described in (**6**). Taking f for the resistance per square inch at a distance of 1 inch from G, which we suppose to coincide with the neutral axis, we have $f \times$ G B for the resistance per inch at the bottom of the beam, and $f \times$ G A for the resistance at the top. And the *moment of resistance* of the upper flange, whose breadth is b, will be

$$f \frac{b}{3} (GA^3 - GC^3),$$

the moment of resistance of the lower flange, whose breadth is B, is

$$f \frac{B}{3} (GB^3 - GD^3),$$

and the moment of resistance of the web CD, whose breadth is t, is

$$f \frac{t}{3} (GC^3 + GD^3).$$

The *moment of the strain* W at the middle of the beam, whose length of bearing is l, supported at each end, is $\frac{1}{4}$ W . l, which must equal the sum of all the moments of resistance above given. Therefore we have the formula

$$W = \frac{4f}{3l} \{B . GB^3 + b . GA^3 - (b-t) GC^3 - (B-t) GD^3\}.$$

Since the utmost tensile strain that cast iron will bear with safety is about 2 tons per square inch of section, we must put $f \times \mathrm{GB} = 2$, and substitute $\frac{2}{\mathrm{GB}}$ for f in the above formula; all the dimensions to be expressed in inches. As an example we will take the beam whose centre of gravity was determined above (6), where $\mathrm{B} = 8$, $b = 3$, $t = 1$, $\mathrm{GB} = \frac{20}{7}$ or $2{\cdot}86$, $\mathrm{GA} = 5{\cdot}64$, $\mathrm{GD} = 1{\cdot}36$, $\mathrm{GC} = 4{\cdot}64$, $f = {\cdot}7$; putting $l = 100$ inches, we find

$$\mathrm{W} = \frac{4}{3} \times \frac{7}{10} \times \frac{350}{100} = 3{\cdot}27 \text{ tons.}$$

If we invert the beam, putting the larger flange at the top, we have $f = {\cdot}35$ or half what it was before, consequently the strength will be one-half of the above.

58. Cast-iron Columns.—The strength of columns of cast-iron has been largely tested by Mr. Hodgkinson, whose researches thereon are detailed in the "Philosophical Transactions" for 1840 and 1857. The power of a column to sustain a vertical pressure depends much upon the proportion which the diameter bears to the length. When the length is not less than 30 times the diameter, the column will only be broken by bending, and the resistance of the material to crushing will not come into play. In long hollow columns of circular section, and having flat ends, the breaking-weight in tons for iron of average quality is

$$\mathrm{W} = 42\,\frac{\mathrm{D}^{3\cdot5} - d^{3\cdot5}}{l^{1\cdot63}},$$

D being the external and d the internal diameter in inches, and l the length in feet. This formula also applies to solid columns, by making $d = 0$.

$$W = \frac{b \cdot c}{b + \frac{3}{4}c}.$$

Take the last example, and let the length be 5 ft. instead of 10 ft.; then we have,

$$b = 42 \frac{128 - 46 \cdot 8}{13 \cdot 8} = 247 \text{ tons};$$

$$c = 49 \times \frac{22}{7} \{2^2 - (\tfrac{3}{2})^2\} = 269 \text{ tons};$$

$$W = \frac{247 \times 269}{247 + 202} = 148 \text{ tons}.$$

When a column is less than 10 diameters in length, it may be considered as breaking only by crushing and not by bending, so that only the crushing-strength of the section has to be taken into account; this is found by multiplying the area of section by the crushing-strength of the metal per square inch, which is about 49 tons for hollow columns, and 39 tons for solid columns.

If a hollow column is not of uniform thickness of metal throughout, its real strength will be only that of the *thinnest* part.

The above formulæ are for columns without discs at top and bottom; when the ends are expanded in the form of discs so as to distribute the pressure over a greater surface, the strength is somewhat greater than that obtained by these rules.

If an iron column is not set perfectly upright it loses a considerable portion of its strength, which will also be the case if it should be thrust out of the perpendicular by any settlement of the building. When a long column is so much out of perpendicular that the diagonal has

become vertical instead of the axis, it is found to have lost two-thirds of its strength.

The permanent safe load supported by a cast-iron column must never exceed one-fourth of the breaking-weight.

It is found that hollow columns of cast-iron are much more liable to fracture when exposed to great heat than solid columns.

60. WROUGHT-IRON.—When pig-iron is slowly melted in a reverberatory furnace, together with a quantity of iron-oxide, and the mass well stirred or "puddled," much of the carbon is removed, as well as some of the other impurities; the metal thus becomes purer and assumes a semi-fluid state; it is then squeezed between rollers and comes out in the form of *wrought-iron*. In order to purify it still further, it is heated to redness, and again rolled, which increases its strength. Its melting point is much higher than that of cast-iron, being about 4,000° Fahr.

Wrought, or malleable iron, possesses a fibrous and non-crystalline texture, and contains very little carbon, not more than 1 part in 300; so that it is tough, and will bear heavy blows without fracture; it will bend considerably before breaking, and withstands the action of fire much better than cast-iron does; the resistance to compression varies from about two-thirds to four-fifths of the resistance to extension. Wrought-iron is not so generally used for columns as cast-iron, but for very long columns the breaking-weight is considerably more in wrought-iron than (with the same dimensions) in cast-iron; the breaking-weight for pillars with flat ends being found by Hodgkinson* to be,

* *Philosophical Transactions*, 1840.

$$W = 134 \frac{d^{3\cdot55}}{l^2}, \text{ in tons.}$$

d being the diameter in inches, l the length in feet. The values of $d^{3\cdot55}$ and l^2 are given in the Table of powers of numbers in the Appendix. For example, to find the breaking-weight of a solid wrought-iron pillar, 4ins. diameter and 10 ft. long; here we have,

$$W = 134 \frac{4^{3\cdot55}}{10^2} = 134 \frac{137\cdot2}{100} = 194 \text{ tons.}$$

The breaking-weight of the same column in cast-iron is

$$W = 42 \frac{4^{3\cdot5}}{10^{1\cdot63}} = 42 \frac{128}{42\cdot7} = 126 \text{ tons.}$$

The constant 134 was obtained from only a small number of experiments, and is probably much too high for the average qualities of wrought-iron.

Rankine* gives the following formula for the breaking-weight in lbs. of long pillars of wrought-iron flat at both ends;

$$W = \frac{36000 \times A}{1 + \frac{1}{3000}\left(\frac{l}{d}\right)^2}$$

where A is the area of section, l the length, and d the diameter of the pillar, all in inches.

For short columns, in which the resistance to crushing comes into action, cast-iron has the advantage over wrought-iron. The same formula can be employed in finding the strength of wrought-iron columns whose length is less than 30 diameters, as was used for short

* *Applied Mechanics.*

cast pillars, only 16 tons must be taken as the crushing-strength per square inch instead of 49, as in cast-iron. If, then, b is the breaking-weight as calculated by the above formula for long pillars, c the crushing-strength of the section, or the area of section multiplied by 16, then the true breaking-weight of a short wrought-iron pillar, in tons, is

$$W = \frac{b \times c}{b + \frac{3}{4}c}.$$

Taking the last example, 5 feet long, we have

$$b = 134 \frac{137 \cdot 2}{25} = 776 \text{ tons};$$

$$c = \frac{22}{7} \times 4 \times 16 = 201;$$

$$W = \frac{776 \times 201}{776 + 150} = 168\tfrac{1}{2} \text{ tons}.$$

The strength of the same column in cast-iron is 282 tons.

61. Beams.—Wrought-iron is extensively employed in the construction of beams of various forms and sizes. For small beams, as joists, or light girders of moderate length, it is generally rolled in one piece, of an I or **T** form of section, according to requirements.

To find the strength of a rolled joist of I section, let I be the moment of inertia of the section, d its total depth; then, as before shown (**12**) the *moment of resistance* at any section is

$$M = S \frac{I}{\frac{1}{2}d},$$

S being the *coefficient of strength* or strain per square inch of section to be determined by experiment. If d_1 is the

depth of the web, or part between the top and bottom flanges; b the breadth of each flange, t the thickness of the web, we have from (**12**),

$$I = \tfrac{1}{12} (bd^3 - (b-t)\, d_1^3).$$

When the beam of length l is loaded in the middle with a weight W, and supported at the ends, we have by (**8**), the moment of the strain equal to $\frac{1}{4}W \cdot l$. Equating this with the above value of M, we get

$$W = \tfrac{2}{3} S \frac{bd^3 - (b-t)\, d_1^3}{d \cdot l}.$$

To determine the value of S we take an example tested by Barlow,* in which $d = 5$ in., $d_1 = 2{\cdot}5$, $l = 54$, $b = 2{\cdot}125$, $t = {\cdot}85$. This beam bore $8\frac{1}{2}$ tons without the elasticity being injured, so that we may take 4 tons as the *safe permanent load.* Substituting these values in the formula, we get,

$$S = \tfrac{3}{2} \frac{W.\, d.\, l}{bd^3 - (b-t)\, d_1^3} = \tfrac{3}{2} \frac{8960 \times 5 \times 54}{265{\cdot}6 - 20} = 14769 \text{ lbs.}$$

or 7 tons per square inch, which is about one-third of the breaking-weight.

Therefore the *safe permanent load* (W) of a rolled joist of I section may be found from the formula,

$$W = 9846 \frac{b \,.\, d^3 - (b-t) d_1^3}{d \,.\, l}.$$

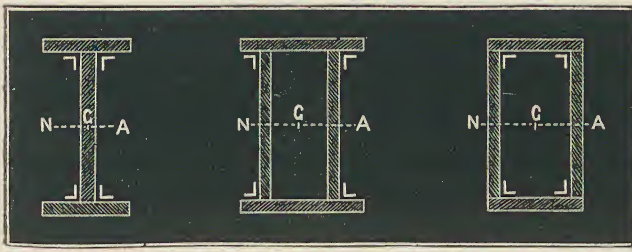
In the foregoing formula all the dimensions are expressed in inches, and the weight in pounds.

Large beams are made by riveting rolled plates together by means of angle-iron, either in the I form of section (fig. 49), or in the box form (figs. 50, 51). The

* Barlow. *On Strength of Materials.* (Lockwood and Co.)

box-girder is found by Fairbairn to be stronger than that having the I form and of the same weight, in the proportion of 100 to 93. If we put b as the breadth of the top

Fig. 49. Fig. 50. Fig. 51.



and bottom plate, d the total depth of the beam in the middle, d_1 the depth between the top and bottom plates, and t the total thickness of the plates forming the web or vertical part; l being the span; (all the dimensions in inches); then the formula for the breaking-weight in the middle is obtained in a similar manner to that for rolled joists, namely:

$$W = 12 \frac{bd^3 - (b - t)\, d_1^3}{d \,.\, l}, \text{ in tons.}$$

The multiplier 12 is derived from an average of the results of Hodgkinson's experiments, as given in the "Report of the Commissioners on the Application of Iron to Railway Purposes." The strength of an I-girder is rather less for the same weight of metal than that of the box-girder, and may be found by multiplying the value of W obtained above by ·93.

Some authorities recommend that the top plate should be somewhat thicker than the bottom, as the resistance

of wrought-iron to extension is rather more than its resistance to compression. But if we compare the results of experiments, we find that no appreciable advantage is obtained in a beam whose top is thicker than its bottom flange, over another beam having equal flanges, where the weights of the two beams are the same.

Let us apply the formula to a box-girder having $b = 24$, $d = 36$, $l = 540$, $t = 2 \times {\cdot}214 = {\cdot}428$, $b - t = 23{\cdot}572$, thickness of top plate $= {\cdot}5625$, thickness of bottom plate $= {\cdot}397$, $d_1 = 35{\cdot}0405$; then we have the breaking-weight,

$$W = 12 \frac{24 \times 36^3 - 23{\cdot}572\,(35{\cdot}0405)^3}{36 \times 540} = 63{\cdot}94 \text{ tons.}$$

The *permanent safe load* should in no case exceed one-third the breaking-weight.

The line NA is the *neutral-axis* passing through the centre of gravity G of the sections, and about which line the moment of inertia is taken.

If the load is uniformly distributed, the beam will bear twice as much as when the weight is concentrated at the centre, as previously shown (**8**). Also when a beam is fixed at one end and loaded at the other, the breaking-weight is one-fourth of that obtained by the formula just given for a beam supported at each end.

It will be seen from the formula that if all the dimensions of a beam are doubled, the value of W is increased four times; if they are trebled, the value of W is increased nine times; and so on. Hence it appears that if the proportions of a beam remain the same, and the actual dimensions are increased, the strength is increased as the second power, or *square*, of the lineal dimensions. This is very nearly borne out by experiments; the aver-

age of which gives 1·9 rather than 2 as the power according to which the strength increases.

In wrought-iron girders the flanges or "*booms*," as they are termed in large beams, are the only parts that have to resist flexure; whilst the web has to resist the *shearing* force (**9**). The resistance to shearing may be considered as equivalent to the resistance to tension. The top flange will be subjected to a compressive force, and the bottom flange to an extending force; now since the resistance to compression is less in wrought-iron than the resistance to extension, it would appear that the area of the upper flange should be greater than that of the lower one in like proportion.

For wrought-iron beams of very large size, such as are required for bridges, the "lattice" principle is generally used. The beams so called are made of section shown in figs. 49, 50, 51, but with the vertical plates replaced by cross-bars of angle and tee-iron riveted to bars of flat iron at their intersections, and also to narrow vertical plates which are themselves riveted to the top and bottom flange, forming the *booms*. Consequently almost the whole of the forces of compression and extension are confined to the booms, as the lattice bars only imperfectly represent the web of solid plates used in the box-girder. Fairbairn considers the strength of this beam to be to that of the box-girder in the proportion of 84 to 100, the weights being equal. The foregoing formula for box-girders may be applied to find the strength of a lattice-girder, by first ascertaining the thickness of vertical plate which would have the same weight as the lattice bars, and calculating the strength of such a box-beam; then the strength of the lattice-beam is found by multiplying the result by ·84. The lattice-girder has an advantage in

exposing a smaller surface to the wind than a solid box-beam, and is therefore preferable for railway bridges. A very full investigation into the strains in lattice-girders will be found in Stoney's "Theory of Strains;" Unwin's "Lectures on Roofs and Bridges;" and in Humber's "Handybook of Strains."

The method of calculating with greater accuracy the strength of large tubular-girders is given in the Report of the Commissioners on the use of Iron for Railway Purposes; also in Tait's "Treatise on the Strength of Materials."

In order that a wrought-iron girder, whose breadth is the same throughout, may have a uniform strength in every part, when the load is equally distributed, the depth must diminish from the centre towards the two ends in the ratio of the ordinates of an ellipse (fig. 48).*

62. Safe-Load.—The same method for determining the load that will produce a given strain per square inch of section at the top or bottom of a wrought-iron beam, may be employed as in the case of a cast-iron beam (**57**, page 136), only here we have GA = GB (fig. 11),† the top and bottom flanges being equal, and GC = GD, so that the moment of strain for a beam loaded at the middle becomes

$$M = \frac{l}{4} W = \frac{f}{12} \left\{ b \cdot D^3 - (b - t) d^3 \right\}$$

where D = AB, d = CD, b = breadth of top or bottom flange, t the thickness of the web; and for wrought-iron we may put $f \times \frac{D}{2} = 5$ tons, for the utmost strain that

* Page 135. † Page 15.

the beam will bear with safety per square inch of section at the bottom or top: or $\frac{10}{D}$ is to be substituted for f in the above formula. In the case of a *box*-girder (figs. 50, 51), t will be the sum of the thicknesses of the two vertical plates forming the web.

In beams which are formed of plates riveted together, the strength of the angle-irons which join the flanges to the web has to be taken into account. Let a be the sum of the areas of section of all the angle-irons, then their resistance w is

$$w = \frac{f \,.\, a \,.\, d^2}{l},$$

which is to be added to the above value of W, to get the true amount that the beam will bear in safety at the middle, all the dimensions being taken in inches, and the load in tons. In the example above given (**61**) of a box-beam, we find from this formula that the load at the middle which produces a strain of 5 tons per inch on the flanges is 19 tons without reckoning the resistance of the angle-irons; if the area of section of the four angle-irons is 12 square inches, they will increase the resistance by $7\frac{1}{2}$ tons, making a total of $26\frac{1}{2}$ tons as the safe-load at the centre of the beam.

We can also apply this method to beams of T section in which there is only one flange. The position of the centre of gravity n (fig. 11) is determined in such a figure at (page 15), g being the centre of the web, m that of the flange, $ng = \frac{a}{a+b} \times mg$, where a is the area of the flange, b that of the web.

Then from the formulæ in (**57**) we have for the moment of the flange whose breadth is b,

$$f\frac{b}{3}(An^3 - Cn^3),$$

and for that of the web whose breadth is t,

$$f\frac{t}{3}(Cn^3 + Dn^3),$$

and the sum of these is equal to $\frac{1}{4}$ W . l; f being determined from the equation

$$f \times An = 5, \text{ or } f = \frac{5}{An}.$$

Taking the example of which the centre of gravity was found (page 16), we have $mg = 3\frac{1}{2}$, $ng = \frac{7}{6}$, $An = \frac{1}{6}$, $Dn = \frac{25}{6}$, $Cn = \frac{11}{6}$, $b = 3$, $t = 1$, $l = 120$ inches, the total depth being 7 inches, and the thickness of metal 1 inch: also, $f = \frac{30}{17}$, therefore

$$W = \frac{30}{17} \cdot \frac{4}{360}\left\{3\left(\frac{17^3 - 11^3}{6^3}\right) + \frac{11^3 + 25^3}{6^3}\right\}$$
$$= 2\cdot5 \text{ tons},$$

as the load at the middle that will produce a strain of 5 tons per square inch at the flange, when the length of the beam is 10 feet.

The following Table shows the safe-load that rolled beams of I section will sustain at the centre, as calculated from the previous formula (page 148), all of them being taken as 10 feet long; and for any other length l, in feet, the safe-load can be determined by multiplying these values of W by $\frac{10}{l}$.

l	b	D	d	t	W
120 ins.	3 ins.	5 ins.	4 ins.	$\frac{3}{8}$ in.	1·15 tons.
,,	4 ,,	$6\frac{1}{4}$,,	$4\frac{3}{4}$,,	$\frac{1}{2}$,,	2·5 ,,
,,	4 ,,	8 ,,	$6\frac{3}{4}$,,	$\frac{5}{8}$,,	3·6 ,,
,,	5 ,,	10 ,,	$8\frac{3}{4}$,,	$\frac{5}{8}$,,	5·8 ,,
,,	6 ,,	12 ,,	$10\frac{1}{4}$,,	$\frac{3}{4}$,,	11·0 ,,

63. Elasticity.—When wrought-iron in the form of bars is subjected to a longitudinal stretching-force, it will sustain a weight of 25 tons for every square inch of section before being torn asunder. This amount may therefore be taken as the breaking-weight of wrought-iron bars which are subjected to no other strain but that of extension in the direction of their length. The extent to which a vertical rod of iron will be stretched by a given weight W (expressed in pounds), suspended from one end is,

$$l = \frac{W \times L}{E \times A},$$

where E is the *modulus of elasticity* (**11**), which, according to Rankine,* is 29,000,000; L being the original length of the bar; and A the area of section.

From this formula it appears that the elongation is directly proportional to the stretching-force, which is, however, only true within certain limits, and will not be true if W exceeds the *limit of elasticity*, or is more than the bar will bear without its elasticity being injured. The elongation is also *directly* proportional to the original length of the bar, and is *inversely* as its area of section. For example, let us find how much an iron rod, 1 inch square and 36 inches long, will be stretched by a weight

* *Applied Mechanics.*

of 10 tons suspended from one end; in this case we have,

$$l = \frac{10 \times 2240 \times 36}{29,000,000 \times 1} = \cdot 0278 \text{ inch.}$$

The experiments which have been made on wrought-iron show that its tensile strength per square inch in round or square *bars* is from 23 to 27 tons; in flat bars from 22 to 26 tons, and, in the form of angle, or tee iron, 21 to 25 tons; in plate iron the strength when the strain is in the direction of the fibres is from 20 to 24 tons, but, if the strain is at right angles to them, it is 17 to 21 tons.

64. Deflexion.—When a wrought-iron beam supported at each end is loaded in the middle, the deflexion produced will be proportional to the amount of the load, so long as it does not exceed one-half the breaking-weight of the beam; but beyond that limit the deflexion increases more rapidly as the load approaches the breaking-weight, in consequence of the elasticity being impaired. It is shown by writers on the strength of materials, that where the beam has equal top and bottom flanges, the deflexion is expressed by the formula,

$$D = \frac{W}{48\ E} \times \frac{l^3}{I} = \frac{W}{4\ E} \times \frac{l^3}{bd^3 - (b - t)\ d_1^3}$$

in which D is the deflection, l the span, W the load, I the moment of inertia of the section about the neutral axis N A (figs. 49, 50, 51).* To obtain the value of E we take a specimen of a rolled iron joist experimented upon by Barlow, the dimensions of which are given in

* Page 145.

page 144. This was found to deflect ·016 in. for every ton of load, so that we can find E in this case from the equation,

$$E = \frac{W \times l^3}{48\ D \times I} = \frac{W\ .\ l^3}{4\ D\ (\ bd^3 - (b - t)\ d_1^3)} = 10{,}030,$$

all the dimensions being in inches, and W in tons. Let us apply this formula to find the deflexion of the box-girder whose breaking-weight we found above (page 146); and let the load in the centre be 33 tons; then,

$$D = \frac{33 \times (540)^3}{40120 \times 105627} = 1{\cdot}23 \text{ inch}.$$

With half this load, or $16\frac{1}{2}$ tons, the deflexion in the middle will be ·615 inch, which amounts to no more than the $\frac{1}{878}$th part of the span; a deflexion which would be quite invisible to the eye; so that we may consider that a permanent load of at least 16 tons may be safely laid on the centre of this beam; and this is about one-fourth of its breaking-weight.

The value of the *modulus* E varies according to the *quality* of the iron used, and can readily be found from experiment, by using the formula given above (**63**).

When the load is uniformly distributed over the entire length of the beam, the deflexion is five-eighths of that produced by the same load placed at the middle. So that the deflexion due to the weight of the beam itself is five-eighths of that which would be produced if its whole weight were concentrated at the middle and the beam itself supposed without weight. Also, in calculating the deflexion of a beam by a given load, we must add five-eighths of the beam's own weight to the load laid on the middle, to get the true strain. In the last example, the

weight of the beam is not considered; but as this amounts to 3 tons, the true deflexion will be that due to $33 + \frac{5}{8}\,3$, or nearly 35 tons. The true value of D is therefore 1·3 inch.

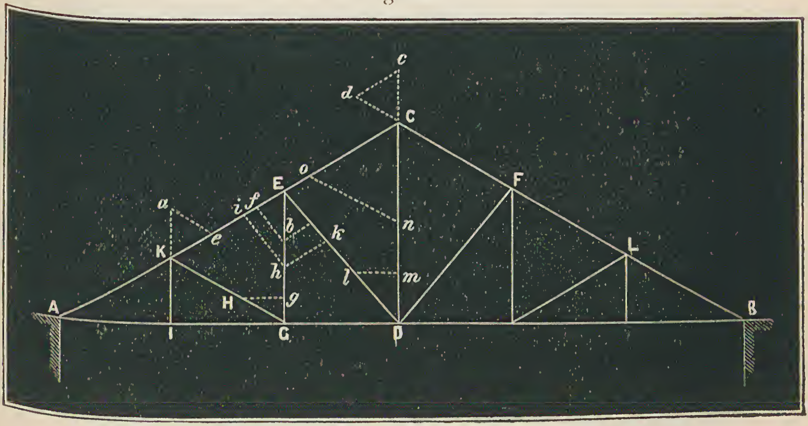
65. Effect of Heat.—When a mass of iron is exposed to alterations of temperature, it continually changes its dimensions, being larger at a high temperature than at a low one; that is to say, it expands in dimensions as the temperature increases. When the temperature rises from 32° Fahrenheit to 212°, cast-iron expands the 900th part of each dimension, its relative proportions remaining the same. Thus a bar of cast-iron 100 feet long will expand 1·36 inch in length while passing from 32° to 212°, or through 180° of temperature; a bar the same length of wrought-iron expands 1·46 in., one of untempered steel expands 1·3 in., and of tempered steel 1·49 in. When iron is used in construction it is exposed in this country to a variation of about 90° during the year, so that a wrought-iron beam 75 feet long will alter above half an inch in length between summer and winter. In cases where it is exposed to the full force of the sun in summer, and the greatest degree of cold in winter, the variation will be considerably more. The actual force developed in expanding a bar of iron by 1·46 inch will be the same as would be required to compress it by that amount, or about 15 tons on the square inch. The cubical expansion is three times the linear expansion. It therefore is absolutely necessary that all large masses of iron should have a certain amount of *play* allowed them, as the expansion and contraction are irresistible forces, and will tear asunder or thrust over any structure that may oppose.

This property of expansion and contraction has been employed for the purpose of drawing together walls which

have been forced out of their perpendicular in opposite directions. Iron bars with nuts and screws being placed tightly across from wall to wall, are heated in the middle, when they expand and loosen the screws which are then tightened up; and as they cool, the contracting force of the bars gradually pulls the walls towards each other. This process being repeated carefully several times, the walls may be brought to their original state of uprightness. When heated to redness, cast-iron is found to lose about one-third of its strength, wrought-iron plates one-fourth, and bar-iron is reduced to one-half its strength.

66. Roofs.—Wrought-iron is much used in the framework of roofs of large span. Fig. 52 represents a common

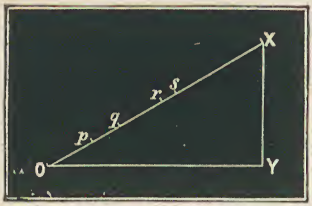
Fig. 52.



form of iron roof. The tie-rod A B being subjected to a stretching force in the direction of its length, and a very slight amount of transverse strain between the several points of support from its own weight, is made of round or flat bar-iron. The rafters AC, BC, having to sustain

both a longitudinal and transverse strain, are made of tee-iron. The struts ED, KG, having to sustain a compressing and transverse strain, are made either of angle or tee-iron; and the vertical rods CD, EG, KI, having only to sustain a stretching force in the direction of their length, are of round iron. Let W be the entire weight which this truss has to sustain; then $\frac{1}{6}$ W is supported *directly* at each of the points K, E, C, F, and L; $\frac{1}{12}$ W *directly* at A and B. Let the lines C*c*, E*b*, K*a*, be equal, and each represent on a scale $\frac{1}{6}$ W. Draw the parallelogram C*c*, making C*d* parallel to BC, and *cd* to AC; then *dc* represents the resolved part of $\frac{1}{6}$ W acting down the rafter AC. Draw OX (fig. 53) parallel to AC, and OY parallel to AB; and take O*p* equal to *cd*. At E (fig. 52) draw the parallelogram E*b*, then E*f* is the resolved part of $\frac{1}{6}$ W acting at E, down the rafter AC; *fb* the resolved part down the strut ED. Take *pq* (fig. 53) equal to E*f*.

Fig. 53.



At K (fig. 52) draw *ae* parallel to KG, then K*e* is the resolved part of $\frac{1}{6}$ W at K down AC, and *ae* is the resolved part down the strut KG. Make *qr* (fig. 53) equal to K*e*. Take GH (fig. 52) equal to *ae*, draw the horizontal H*g*, then G*g* (= $\frac{1}{2}$ K*a*) is the vertical strain on EG from the strut KG; take *bh* = G*g*, and draw *hi* parallel to ED; then *if* is the resolved part in the direction of AC; take *rs* (fig. 53) equal to *if*. E*k* (fig. 52) represents the compression on the strut ED, which is conveyed to D; take D*l* = E*k*, and draw *lm* horizontal; then twice D*m* represents the vertical strain on CD caused by the two struts ED, FD. Take C*n* = 2 D*m*, draw *on*

parallel to CB; then *Co* is the compression down AC produced by the rod CD. Take *s*X (fig. 53) equal to *Co*; then the line OX represents the total compression down AC on the same scale that *Cc* represents $\frac{1}{6}$ W. Draw the perpendicular YX (fig. 53), then OY represents the horizontal strain on the tie-rod.

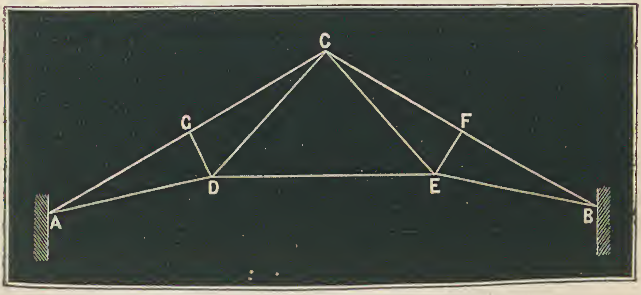
By this means the strains on the several pieces of the framework can be readily found by measuring with a scale.

One half the weight of the tie-rod is sustained by the five vertical rods, so that each has one-tenth of the weight of the tie-rod added to the other strains. This is, however, generally of but small amount.

The analytical methods of finding the strains on the several parts of a roof, will be found in Fenwick's "Mechanics of Construction," and Rankine's "Applied Mechanics."

67. Professor Maxwell's method of diagrams, which

Fig. 54.

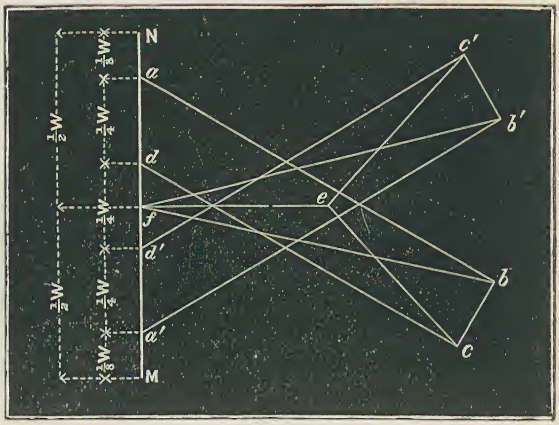


has been previously described (**3**), enables us readily to find the relative strains on the different parts of an iron roof. The following is an example of its application to a common form of roof (fig. 54). Let W

represent the total vertical weight which the truss has to bear; then $\frac{1}{8}$ W is *directly* supported at A and B; $\frac{1}{4}$ W at G, C and F. The vertical reaction at A and B is $\frac{1}{2}$ W.

Now draw a vertical line M N (fig. 55) whose length

Fig. 55.



represents the weight W on any convenient scale; bisect M N in *f*. Take N*a*, M*a′*, *fd*, *fd′*, each equal (on the above scale) to $\frac{1}{8}$ W.

Draw *ab* and *dc* parallel to B C (fig. 54); *fb* parallel to B E; *bc* parallel to E F; *ce* parallel to E C; *fe* parallel to D E; and so also for the opposite side. Then, if we measure the lengths of the several lines thus drawn on the above scale, we obtain the strains in the several parts of the roof to which they are respectively parallel.

Thus, *f*N represents the reaction at A or B, and equals $\frac{1}{2}$ W; N*a* the load supported at the joints A or B, and equals $\frac{1}{8}$ W; *ad* the vertical pressure at G or F, and

equals $\frac{1}{4}$ W; dd' the vertical pressure at C, and equals $\frac{1}{4}$ W; ab the strain down AG or BF; dc the strain down CG or CF; bc the strain along EF or GD; bf the strain along BE or AD; ec the strain along EC or DC; fe the strain along DE. Several examples of the application of this method will be found in Unwin's "Lectures on Iron Roofs and Bridges."

68. Circular Roof.—Semi-circular ribs of wrought-iron are frequently used to carry the load in cases of roofs of wide span; and for the investigation of their thrust and strength we can use the same method as we adopted for ribs of wood (**52**). Supposing as before that the rib is loaded by means of seven purlins, one of which is at the crown of the arch, we take w as the load at each purlin, and W the weight of the half-rib acting at its centre of gravity G (fig. 44, page 124), F being the horizontal thrust at the crown from the other half-rib, we have to find its value by taking moments of all the forces about the point A at the springing; so that we have,

$$F \times OB = W \times Ag + w\left(\tfrac{1}{2}\, OA + Ae + Ad + Ac\right).$$

We can now consider F as the thrust of the rib acting horizontally at A; and, in order to prevent F from overturning the wall AZ on which the roof rests, the moment of F about Z must not exceed the sum of the moments of the weight of the roof and of the wall taken about the same point. If Q is the weight, and t the thickness of the wall, we can determine the value of t that will produce equilibrium between the moments, from the equation

$$\left(Q + W + \frac{7}{2}\, w\right)\frac{t}{2} = F \times AZ.$$

In order to find the strength of the rib, we have to take

moments about the point H, where the strain is greatest; the moment of the strain at H is

$$M = w\left(\tfrac{1}{2}\,Hh + Hp + Hq\right),$$

and from (**62**) we have for a beam of I section

$$M = \frac{5}{6D}\left\{b\,.\,D^3 - (b - t)\,d^3\right\} \times 2240$$

the loads being expressed in lbs., and the strain at the flanges being 5 tons per square inch.

As an example, we take a rib having a span of 40 feet, its depth being 12 inches, and breadth 6 inches, the section being that of the last example given in the Table (**62**, page 151), its weight being 56 lbs. per lineal foot, so that $W = 1804$ lbs.; also let $w = 4000$ lbs. Then we have from the figure (page 124), $OG = 18{\cdot}45$, $Og = 13$, $Ag = 8$, $OA = 21$, $Ae = 13{\cdot}15$, $Ad = 6{\cdot}5$, $Ac = 2$, $OB = 20{\cdot}5$; hence we find by putting these values in the first equation, $F = 6977$ lbs. Suppose the wall to be 12 feet long, and 20 feet high, its weight per cubic foot being 120 lbs.; then $Q = 28800\,t$, and the second equation becomes

$$28800\,\frac{t^2}{2} + 15804\,\frac{t}{2} = 6977 \times 20$$

from which we find that $t = 4$ feet, which is the thickness of the wall for equilibrium only. If the wall is only 1 foot long, then we find $t = 8$ feet, but the quantity of walling is one-sixth of what it was in the former case.

The moment of the strain at H is

$$\begin{aligned} w \times l &= 4000\,(8{\cdot}5 + 9{\cdot}15 + 2{\cdot}5) \times 12 \\ &= 967200, \end{aligned}$$

and the moment of resistance of the section is

$$M = \frac{5}{72} \times 4714 \times 2240 = 732945$$

which is less than the moment of strain in the proportion of 3 : 4, so that for safety we must increase the strength at H by one-third, or else reduce the load w to 3000 lbs. The strain, however, at H, is about double that at the crown under this mode of loading the beam, so that we may reduce the strength at the crown to half that at the haunches. The breadth b of the beam at H for any given value of D, d, t and w, can be determined from the equation

$$b = \frac{\frac{72}{5}\, w \,.\, l - t \,.\, d^3}{D^3 - d^3}$$

w being in tons, and all the dimensions in inches.

In the above example where $w = 4000$ lbs. we find that with a depth of 12 inches the breadth of the flanges for safety should be 8·3 inches.

69. STEEL.—Where great strength combined with lightness is required, steel is the material which is most useful. The strength of steel depends on the proportion of pure carbon which it contains, varying in different qualities from three-eighths to one-and-a-half per cent. In wrought-iron the quantity of carbon is from one-eighth to one-half per cent; and in cast-iron from two-and-a-half to five per cent.

From experiments made by Mr. Vickers of Sheffield,* it appears that the *tensile* strength of steel is increased by the addition of carbon until the proportion reaches $1\frac{1}{4}$ per

* Paper read at Institution of Mechanical Engineers, 1861.

M

cent., the breaking strain being then as high as 69 tons per square inch. Beyond this degree of carbonization, the steel becomes gradually weaker until it reaches the form of cast-iron, and will sustain a tensile strain of only 6 or 6½ tons per square inch.

For sustaining blows, however, it is found that the steel cannot contain too little carbon; any increase in the quantity of carbon being accompanied by an increase of brittleness. It is therefore necessary in practice to obtain a material which shall combine the power of resisting a tolerably high tensile as well as transverse strain; and these qualities are found in steel that contains five-eighths to three-fourths per cent. of carbon, and in which the tensile strength is from 45 to 50 tons per square inch of section; the specific gravity being 7·85.

Experiments by Major Wade (U.S. Army) on small cast-steel cylinders, whose length was about 2½ diameters, gave the crushing strength of steel, not hardened, at 89 tons per square inch of section; and for hardened steel of "mean temper," 175 tons per inch. The crushing strength of steel of "high" or of "low" temper was rather less than that of the "mean."

A large number of experiments were made by Fairbairn on steel of various qualities and from several different manufacturers.* The *modulus of elasticity* from thirty of the best specimens averaged 31,000,000; whilst in some of the inferior qualities it was only two-thirds of that value. The average tensile strength of thirty of the best specimens was found to be 47·7 tons per square inch, or about double that of hammered iron. The resistance to compression was about double the resistance to extension.

* *Report of British Association*, 1867.

Hence it follows that a horizontal steel beam having a top and bottom flange, and being strained transversely, should have the area of section of the bottom flange made double that of the top flange.

It is found that when a steel beam is subjected to a transverse strain, the strain corresponding to the *elastic limit* approaches more nearly the breaking-weight than it does in iron; and consequently that a steel beam may be over-loaded with greater safety than an iron one.

Numerous experiments on the strength of steel of various kinds when subjected to a pulling, thrusting, shearing, or bending strain, have been made by Mr. Kirkaldy, some of which are described in his work on the "Results of Experimental Inquiry into the Mechanical Properties of Steel." From these it appears that Swedish steel has a tensile strength varying from 28 to 50 tons per square inch, and a crushing strength varying from 45 to 82 tons per square inch.

The strength of a long steel pillar may be approximately found from the equation

$$W = 100 \frac{d^4}{l^2}$$

where W is the breaking-weight in tons, d the diameter in inches, l the length in feet.

70. Preservation of Iron from rust is a matter of great importance, as the strong affinity which it has for oxygen renders it peculiarly liable to rapid decay when exposed to damp air, especially when combined with high temperature, which greatly accelerates corrosion. Freshly-made castings are protected for a time from rust by being coated with linseed oil, then smoked over a wood fire,

and afterwards dipped in turpentine. Paints of various kinds are employed to preserve iron from rust, these being compounded either of some metallic oxide or carbonate with oil and turpentine, or else in the form of a varnish composed of a resinous gum with either oil, turpentine, or some other spirit. The surface of the iron must be thoroughly cleansed from all rust before the paint is applied, otherwise it will oxidise under the paint, and throw off the coating. All these coatings, however, only act temporarily, and require to be frequently renewed, and therefore cannot be looked upon as permanent protections. The process called "galvanising" has been extensively used for the protection of iron, and consists in giving it a thin coating of zinc, the iron being placed in a bath of melted zinc, the surface of which is covered with sal-ammoniac, by which the zinc-oxide is dissolved. The zinc becomes alloyed with the iron, and, although itself soon oxidised by contact with air, yet the oxide thus formed protects the zinc from further corrosion, and the iron is protected by the zinc as long as that lasts, which may be for several years. Another mode of protecting iron is by forming an *enamel* on its surface, which, after being well cleaned with sand and dilute oil of vitriol, is brushed over with a compound of silica, borax, feldspar, kaolin, and water; dry feldspar, soda, borax, and a metallic oxide, are then sprinkled over it in a finely-divided state, and the iron is then heated until these materials are melted into an enamel which firmly adheres to the surface.

Another process of recent discovery, and one which appears to give a more permanent protection to iron than any of the foregoing, is the formation of the black or magnetic oxide on the surface by passing superheated

steam over the metal for several hours when red-hot; the oxide thus formed is harder than the metal itself, and adheres firmly to the surface, which it renders black, but, at the same time, prevents the formation of rust on the surface which is thus oxidised.

CHAPTER VII.

FLUIDS AT REST AND IN MOTION.

71. A FLUID may be defined as a substance which can be divided in any direction, and the particles of which can be moved about amongst each other by the least conceivable force; the particles of a fluid transmitting pressures to each other equally in all directions.

Fluids are divided into two great classes, namely, *liquids* and *gases;* the former, of which water is a type, being called incompressible or inelastic fluids; and the latter, of which air may be taken as the type, being called elastic fluids. In liquids there is an attraction between the particles which causes them to collect in the form of drops; but with gases there is a repulsion between the particles which gives them a tendency to expand to an almost indefinite extent. While gases can be readily compressed to any amount that may be required, liquids require an enormous force to reduce their volume in any perceptible degree; thus the effect of the pressure of 15 lbs. on the square inch on water enclosed in a vessel is only to compress it by less than $\frac{1}{20000}$ of its volume, so that for all practical purposes we may consider water as incompressible.

The condition in which bodies exist, whether as solids, liquids, or gases, depends to a great extent on temperature. Thus we find water as a solid in the form of ice at

or below 32° Fahr., as a liquid between 32° and 212°, and in the gaseous form of steam above the temperature of 212°. The same may be said of the metals, which are solid at ordinary temperatures, but which by the application of sufficient heat can be liquefied, and by greater heat still can be converted into gases. Many substances, also, which are gaseous at ordinary temperatures, can by the combined action of intense cold and heavy pressure be brought to the liquid or even solid condition. The only fluids, however, which we shall consider in this chapter are *Water* and *Air*.

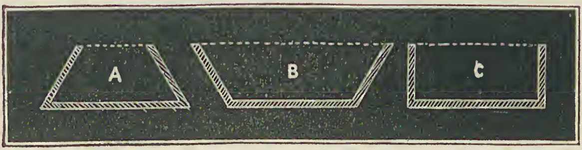
72. WATER.—This substance was formerly considered as an *element*, but is now known to be a chemical combination of the gases oxygen and hydrogen, and to this composition is owing the power it possesses of acting upon metals and other solid bodies, which decompose it and absorb the oxygen. This process is familiar to us in the corrosion of iron by exposure to moisture, the action being facilitated by rise of temperature. Water, as before mentioned, can be obtained either in the solid, liquid, or gaseous state, simply by a change of temperature; and it is the only substance to which we give a different name in each of these conditions, although its chemical composition remains unaltered. Water possesses the peculiar property, found also in a few other substances, of being heavier or denser in the liquid state than in the solid when the temperatures are nearly the same, so that the solid water or ice floats on the surface of the liquid. The highest temperature to which liquid water can be raised in an open vessel is about 212° Fahr. with the barometer at 30 inches, which is called its boiling point, and is also that of its minimum density as a liquid. As the temperature falls below 212° the

density increases and the liquid contracts in volume until it arrives at the temperature of 39·2° Fahr. when it has attained its maximum density, 1,000 cubic inches at 212°, having contracted to 958 inches at 39·2°. If the temperature is lowered still further it begins to increase in volume, its density decreasing, until it reaches 32° Fahr., when it suddenly expands by one-tenth of its bulk and solidifies in the form of ice, a force of many tons per square inch being exerted in the expansion upon the sides of any vessel in which it is contained. This is the cause of the bursting of water pipes which are full of water when a sudden frost occurs, pieces of ice being often forced through the opening made in a lead pipe by the expansive force developed; such pipes being generally made to withstand a pressure from within of not more than 150 lbs. per square inch of surface. Blocks of stone which have been newly quarried are also readily split by the freezing of the moisture which they contain.

73. Water in Tanks.—The pressure of water on the surface of any part of a vessel which contains it, is a weight equal to that of a column of water whose base is the area of the surface immersed, and whose height is the depth of its centre of gravity below the top of the water. Thus, for example, the pressure on the bottom (supposed to be level) of a tank full of water is the weight of a column of water whose base is the area of the bottom, and height the depth of the tank; and it is independent of the form of the tank, being the same whether the sides are sloping or vertical. Thus, in the three tanks A, B, and C (fig. 56), having equal bases and the same depth, the pressure on the bottom will be the same in all when filled with water. The pressure of the water on the *side* of a full tank, whether vertical or inclined, is found

by multiplying the area in feet of its surface by the depth of its centre of gravity below the top of the tank, also in feet, and the result by the weight of a cubic foot of water, or $62\frac{1}{2}$ lbs. If the side is rectangular, the centre of

Fig. 56.



gravity is at a depth equal to half that of the tank; so that the pressure on each side of a cubical vessel is half that on its bottom. If l and d are the length and depth of a side in feet, and P the pressure in lbs. on the side, we have

$$P = 31{\cdot}25 l d^2.$$

In the case of a cubical tank 9 ft. each way, filled with water, P will be 10 tons.

The pressure on the *side* of a tank is also quite independent of the area of surface which the *bottom* of the tank covers, being the same for a narrow vessel as for one of unlimited extent; provided only that the area of the side itself and the *depth* of the water remain unaltered.

When a tank has sloping sides, which are wider at the top than at the bottom, the position of the centre of gravity of any side must be determined as in (6). Let d be the vertical depth of the tank, h the height of a side measured on the slope, a the breadth at top, b the breadth at bottom; then the pressure on the side, when the tank is full, is that of a column of water whose base is $\frac{1}{2}(a+b)\,h$, and whose height is $\frac{d}{3} \times \frac{a+2b}{a+b}$.

74. Centre of Pressure.—When a rigid plane is pressed upon by water in which it is immersed, there is one point in that plane at which the resultant of all the pressures acts; and this point is called the "centre of pressure." If, therefore, a force equal to the resultant is applied at the *centre of pressure*, all the pressures will be counterbalanced, and the plane will be in equilibrium.

The position of the centre of pressure can only be obtained by help of the higher mathematics, the methods of which will be found in any analytical treatise on hydrostatics. The position of the *centre of pressure* in a rectangular plane immersed in water is shown to be at one-third of its height from the base, measured up the centre-line. Hence it follows that when a tie-rod is placed across from side to side of a tank having vertical sides and filled with water, its position must be in the middle of the side at one-third of its depth from the bottom. In the case of the cubical tank above mentioned having sides 9 ft. each way, the centre of pressure will be 3 ft. from the bottom, and the iron tie-rod placed across should have a section of 2 inches to allow a strain of not more than 5 tons per inch.

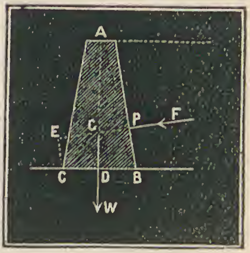
In a plane surface of which the top and bottom are parallel, but of different widths, where a is the width at top, b that at bottom, and h the height or distance between the top and bottom, the distance of the *centre of pressure* from the top is

$$\frac{h}{2} \times \frac{a + 3\,b}{a + 2\,b};$$

and if $a = 2b$, the distance of the centre of pressure from the top is $\frac{5}{8}\,h$; the distance being in all cases measured on the centre line at right angles to the base.

Let ABC (fig. 57) be the section of the wall of a reservoir full of water; F the pressure of the water acting perpendicularly to AB at P, the *centre of pressure*. Let W be the weight of the wall acting vertically at G, its centre of gravity. Draw EC perpendicular to the direction of the force F; then, in order that the wall may not be overthrown, we must have F × EC less than W × DC.

Fig. 57.



If the section of the wall is rectangular, and the face next the water vertical, we can determine the thickness t which must be given to it in order to insure its stability, as in the case of retaining walls (Chap. II.), by taking moments about a point S (fig. 24, page 36), at a distance $\frac{1}{8}t$ within the outer edge C of the wall. Putting h for the height of the wall, w for the weight of a cubic foot of water, W the weight of a piece of wall one foot in length, whose weight per cubic foot is w'; we have

$$F = \tfrac{1}{2}\, w h^2$$
$$W \times DS = \tfrac{3}{8}\, w' h t^2$$
$$F \times EC = \tfrac{1}{6}\, w h^3$$

Equating these moments, we obtain t, the thickness of the wall, or

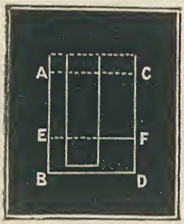
$$t = \tfrac{2}{3}\sqrt{\frac{w}{w'}} \,.\, h$$

Let the wall be of solid concrete, where $w' = 140$ lbs., then since $w = 62{\cdot}5$ lbs., we get, in this case,

$$t = {\cdot}445\, h.$$

75. Hydrostatic Pressure.—When vessels of different sizes are placed on the same level, and made to communicate by a pipe at the bottom, water poured into one vessel will rise to the same level in all. Thus, let the small vessel (fig. 58) AB be filled up to A, the water will rise in the larger one CD to C, and the line AC will be horizontal; so that the weight of the small quantity of water in AB balances that of the larger quantity in CD. Suppose the vessel CD be closed tightly by a plug at any point F, and let AB be filled up to A with water. Draw the horizontal line EF; then the column of water AE exerts a pressure upwards on the underside of the plug at F. If A is the area of section in square inches of the larger vessel, a that of the smaller, p the pressure per square inch of the column AE; then $p \times a$ is the pressure at E of the column AE, and $p \times A$ the pressure upwards of the plug at F; or the pressure at F is to that at E as $p \,.\, A$ to $p \,.\, a$, or as A to a. Therefore the pressure on the underside of the plug at F $= \dfrac{A}{a} \times$ weight of column AE.

Fig. 58.



The column of water AE is called the *head of water* due to the height EA above the plug at F. Hence we see that the pressure upwards on the plug is increased by increasing A, the area of the larger vessel, or by diminishing the area a of the smaller one, provided that the height of the column AE is increased in proportion as a is diminished; so that a very small quantity of water may produce a very great pressure on a large surface. This principle is applied to the construction of hydraulic lifts; a piston being substituted for the plug in

the larger vessel, which is raised by the pressure of water from a small tube carried up to a tank in the highest part of the building. If the area of the piston is 50 times that of the section of the tube, the pressure upwards upon it will equal 50 times the weight of the column AE; so that it will not only be made to ascend, but will also be capable of raising a heavy weight with it, until equilibrium is established by the gradual decrease of the head of water, or of the height AE.

The pressure on the piston A at F is, however, independent of the area a of the tube, for if h is the height AE, we have pressure at $\mathrm{F} = \frac{\mathrm{A}}{a} \times$ weight of column AE, $= \frac{\mathrm{A}}{a} wha = \mathrm{A}wh$, which is increased by increase of A and h.

We have a familiar example of the practical application of the above principle in the means adopted for the supply of hot water throughout every story of a house from a boiler in the basement. A cold water tank C is provided at the top of the house, and a pipe therefrom conveys water to a small closed boiler in the lowest story, in which it is heated, and passes by another pipe from the boiler to any other part of the house, owing to the *head of water* which is due to the height of C above the boiler. It is usual, however, to provide a second tank H for hot water, and this must be entirely closed-in to prevent the pressure from C causing the water to overflow and escape. The tank H may be placed in any convenient part of the house, but it is best to have it as near the boiler as possible so as to avoid the risk of its being frozen up in winter. The object of having the tank H is to obviate the necessity of having a large boiler, and to

contain a reserve of hot water ready for use; it should be provided with a safety-valve, which must, however, be weighted with a greater pressure than that due to the head of water from C, which will be 15 lbs. per square inch for every 34 feet of height, so as to prevent an escape of water from the valve: the object of such a valve being to prevent an undue accumulation of steam, and to prevent the bursting of the boiler in case the pipe from C should happen to become stopped up.

76. Specific Gravity.—When a heavy substance is placed in one scale of a pair of balances, it is said to have the same weight as another heavy body placed in the opposite scale, when they exactly balance. If the heavy substance, instead of being placed in the scale, is suspended from the bottom of it in a vessel of water, it is now found that the body in the opposite scale more than balances it, and that it has lost a certain amount of weight. This loss of weight is due to the upward pressure of the water upon the body, which is equal to the weight of water the body displaces, or to a quantity of water equal in bulk to that of the heavy body. Let w be the weight of the body in air, w' its weight in water; then $w - w'$ is the weight of an equal bulk of water. The proportion which the weight of any body bears to the weight of an equal bulk of water is called its "specific gravity." The *specific gravity*, therefore, of a body is represented by the ratio

$$\frac{w}{w - w'}$$

the value of which has been carefully determined for nearly all known substances by different experimenters, and a copious Table is given in the Appendix of the

specific gravity (that of water being taken as 1000) and weight per cubic foot of most of those employed for building purposes, it being often a very useful guide as to the quality of a material to know its specific gravity, since the density or compactness is in proportion thereto. In selecting a building stone from several specimens whose chemical composition may be similar, it will generally be found that the stone having the highest specific gravity is the best able to resist the action of weather.

If we take the weight of a cubic foot of water as 1,000 oz. or $62\frac{1}{2}$ lbs., we have only to multiply the specific gravity of a body by $62\frac{1}{2}$ lbs. and we find the weight per cubic foot of that substance. We are also enabled to detect any unsoundness in a mass of cast metal which may appear externally to be perfect, but, from cooling too rapidly, has become honey-combed in the interior, as its specific gravity will in that case be less than for a solid mass of the same metal. The impurities and adulterations of metals and other materials can be detected by finding their specific gravity; and also the proportion in which the two metals in any alloy are mixed can be determined. Brass, being a mixture of copper and tin, has a specific gravity between those of the two metals; the specific gravity of copper is 8·9, and of tin, 7·29; that of brass varies from 7·8 to 8·4, according to the proportions in which these metals are mixed. The specific gravity of an alloy of two metals may be found when we know the specific gravities S and s of its constituents, and the volumes V and v of each, for we have

$$\text{Specific gravity of alloy} = \frac{VS + vs}{V + v}$$

To find the specific gravity of wood, or of any substance lighter than water, let it be made to sink by attaching a heavy body whose weight in water is w_1; and let w' be the weight of the two bodies in water, then the specific-gravity required is

$$\frac{w}{w + w_1 - w'}.$$

It often happens that a baulk of timber is rotten in the middle, from the pith having decayed (**40**), by which its value when cut up is much diminished; this can be at once detected by finding the specific gravity of the baulk and comparing it with that of a piece of the solid wood.

77. Water in Motion.—When water is allowed to flow out of a tank through a small orifice at the bottom, the height (h) of the water in the tank being kept at a constant level, the velocity V of the efflux in feet per second is expressed by the formula

$$V = \sqrt{2g} \cdot \sqrt{h} = 8\sqrt{h},$$

where g is the force of gravity, say 32 feet per second, as explained in Chapter I. (**1**), and h is the height in feet *due* to the velocity V. The quantity h is also called the *head of water*, and the velocities of efflux for different *heads* are to one another as the *square roots* of their respective heads. If the tank is allowed to empty itself through the orifice, no fresh water being supplied to make up for the loss, the velocity will gradually diminish with the decrease of *head*, so that the time of emptying the tank will be twice as great as that required for discharging an equal volume of water when it was kept full.

The quantity of water discharged per second can easily be calculated from the formula for V when the area of

the orifice is known; allowance, however, has to be made for the contraction of the stream in passing through the orifice, its actual, or *effective*, area of section being only about ·6 or $\frac{3}{5}$ths of that of the orifice. Putting a for the *effective* area of the stream at the orifice in square feet, Q the volume in cubic feet of water discharged per second, we have

$$Q = V \cdot a = 8a\sqrt{h}$$

and if we multiply Q by $6\frac{1}{4}$ we obtain the number of gallons discharged in every second of time.

78. Water in Pipes.—When water is conveyed from the bottom of a reservoir by means of a long horizontal pipe, it is found that the surface of the pipe offers a certain resistance to the passage of the water, thereby retarding its velocity, so that the effect of the *head* of water is diminished, and the discharge is therefore much less than that obtained above. It has been shown by experiment that this resistance is independent of the height of the *head* or the pressure of the water on the sides of the pipe, but is nearly as the square of the velocity at which the water is passing along the pipe, and is also proportional to the surface of the pipe itself which is exposed to the action of the water; so that as the surface of a circular cylinder is the smallest for a given area of section, it is advisable to make pipes and channels for conveying water as nearly circular as possible, in order to reduce their resistance to a minimum. The term friction is usually applied to this resistance, but it is very different to the friction of solids as described in section (**13**).

If we call f the co-efficient of friction which is determined by experiment, l the length and d the diameter of the pipe, in feet; v the velocity in feet per second; g the

force of gravity, or 32·2 in this latitude; then δ, the *loss of head*, is found to be nearly,

$$\delta = f\frac{l}{d}\cdot\frac{v^2}{2g}, \text{ in feet,}$$

$$= \frac{f}{2g}\cdot\frac{l}{d}\cdot v^2$$

$$= \cdot 0004\,\frac{l}{d}\,v^2, \text{ nearly.}$$

The value of f is not, however, quite constant, being greater for low than for high velocities; its value has been calculated by different experimenters whose results show considerable divergence, but the following is the value given by Weisbach in his "Mechanics of Engineering"—

$$f = \cdot 01439 + \frac{\cdot 017155}{\sqrt{v}}.$$

From this we find that when

$v =$ 1 ft. per sec.,	$f = \cdot 0315$,	$\frac{2g}{f} = 2032$,	$\sqrt{\frac{2g}{f}} = 45$
$v =$ 2 ,,	$f = \cdot 0265$,	,, $= 2421$,	,, $= 49$
$v =$ 3 ,,	$f = \cdot 0243$,	,, $= 2640$,	,, $= 51$
$v =$ 6 ,,	$f = \cdot 0214$,	,, $= 3009$,	,, $= 55$
$v =$ 12 ,,	$f = \cdot 0193$,	,, $= 3337$,	,, $= 58$
$v =$ 20 ,,	$f = \cdot 0182$,	,, $= 3538$,	,, $= 59$

For example, suppose a pipe, 8 in. diameter and 2,000 ft. long, conveys water at a velocity of 3 ft. per second; the *loss of head* will be,

$$\delta = \cdot 0243\;\frac{2000}{\frac{2}{3}}\cdot\frac{9}{64\cdot 4} = 10\cdot 23 \text{ feet.}$$

It appears from the formula that in order to obtain the least possible *loss of head* in leading a quantity of water, the pipe must be made as wide as possible and not unnecessarily long.

If Q is the discharge, F the transverse section of the pipe $= \pi \frac{d^2}{4}$, or, $\cdot 7854 d^2$, when the section is a circle,

$$\text{then, } v = \frac{Q}{F} = \frac{4Q}{\pi d^2} = 1{\cdot}273 \frac{Q}{d^2}$$

Substitute this value for v in the formula for *loss of head*, and we have,

$$\delta = \frac{f}{2g} \left(\frac{4}{\pi}\right)^2 \frac{l \,.\, Q^2}{d^5};$$

from which it is seen that if all the other quantities remain unaltered, and $2d$ is put for d, then the value of δ must be divided by 2^5 or 32.

This last formula tells us how much *fall* we must give to a pipe, in order that it may convey a given quantity Q from one place to another. In the previous example, if a *fall* of 10·23 feet is required to convey a given quantity in a pipe of 8 inches diameter, then a *fall* of only $\frac{10{\cdot}23}{32}$, or ·32 feet, is required to produce the same discharge from a tube 16 inches diameter.

To find the velocity v, in feet per second, of the discharge of water from long tubes, where h is the *head* in feet, we have

$$v = \sqrt{\frac{2g \,.\, h \,.\, d}{f \,.\, l}} = \sqrt{\frac{2g}{f}} \sqrt{\frac{d}{l}} \sqrt{h} = 50 \sqrt{\frac{d}{l}} \sqrt{h},$$

nearly,

$$\text{and, } Q = F \,.\, v = \frac{\pi d^2}{4} v$$
$$= \cdot 7854 d^2 \,.\, v.$$

In all cases the pipes are supposed to be running full.

Let us compare this value of v with that of V obtained above (**77**) for water issuing from an orifice at the bottom of a tank; let $d = \frac{2}{3}$, $l = 2000$; then we have

$$V = 8 \sqrt{h}$$
$$v = \tfrac{10}{11} \sqrt{h}$$
$$V : v :: 88 : 10$$

and the values of Q, or the volumes discharged in a given time will be in the same ratio.

79. Bends and Elbows.—A considerable loss of head may be produced by changes in the size and direction of a pipe conveying water. Any sudden enlargement or contraction of the diameter produces a sudden change of velocity and a consequent loss of head; but this loss may be reduced to a very small amount if the change in the size of the pipe is made gradually. The greatest loss of head is occasioned by sudden turns in the direction of the pipe, in the case of a sharp elbow the loss depends on the angle of deviation, and can be expressed by the formula,

$$\delta = k \frac{v^2}{2g}$$

k being the co-efficient of resistance to be determined by experiment for different angles of deviation from the straight line. By experiments on a pipe $1\frac{1}{4}$ inch in diameter, Weisbach found that when the angle of deviation was 20°, $k = \cdot 046$; for the angle of 40°, $k = \cdot 139$; for 60°, $k = \cdot 364$; for 90°, $k = \cdot 984$; so that with an

elbow forming a right angle, the loss of head is nearly as much as the height due to the velocity. With pipes of smaller diameter the value of k was found to be much greater.

Bends formed by arcs of circles cause much less resistance to the passage of water than elbows do, but they also cause a partial contraction of the diameter of the stream which reduces the head; but this loss may be reduced by gradually enlarging the cross section of the pipe at the middle of the bend. The co-efficient of resistance k in this case depends on the ratio of the radius of the section of the pipe to the radius of curvature of the bend. Thus, in pipes of circular section having a for the radius of the pipe, and r for that of the bend, Weisbach obtained the following values of k from which the loss of head δ can be calculated in the formula,

$$\delta = k \frac{v^2}{2g}$$

$$\begin{array}{rcrcr}
\frac{a}{r} = & \cdot 1, & k = & \cdot 131 \\
,, = & \cdot 2, & ,, = & \cdot 138 \\
,, = & \cdot 3, & ,, = & \cdot 158 \\
,, = & \cdot 4, & ,, = & \cdot 206 \\
,, = & \cdot 5, & ,, = & \cdot 294 \\
,, = & \cdot 6, & ,, = & \cdot 440 \\
,, = & \cdot 7, & ,, = & \cdot 661 \\
,, = & \cdot 8, & ,, = & \cdot 977 \\
,, = & \cdot 9, & ,, = & 1 \cdot 408 \\
,, = & 1 \cdot 0, & ,, = & 1 \cdot 978
\end{array}$$

In drain-pipes for the conveyance of sewage-water, elbows and bends are frequently the cause of their being choked up, as the stoppage or reduction of the velocity at

those points causes a deposit of the solid matters which are being carried along to take place, whereby the transverse section is reduced, and a still further obstruction is caused which in time results in a complete blocking up of the pipe.

80. Strength of Water-pipes.—Pipes for conveying water should be capable of sustaining a pressure equal to not less than 10 atmospheres, or 150 lbs. on every square inch of surface, which is equivalent to a head of water 340 feet high. The thickness therefore of similar pipes must be proportional to their diameters, as well as to the head of water or pressure per unit of surface. For pipes of diameter d, the following are the values of t, the thickness of the pipe for a pressure of 10 atmospheres, as found by Weisbach's experiments on pipes of different materials:

Sheet-iron, $t = \cdot 0086\, d + \cdot 12$, inches.
Cast-iron, $t = \cdot 0238\, d + \cdot 34$, ,,
Lead, $t = \cdot 0507\, d + \cdot 21$, ,,

81. Air.—The atmosphere surrounding the earth is found to consist of a mixture of the two gases, oxygen and nitrogen, the proportions of which are very nearly the same, whether the air is on the top of a mountain or down in a valley. 100 parts by weight of air contain 23 parts of oxygen and 77 of nitrogen, and as the former is the heavier of the two in the ratio of 16 to 14, the proportion of the gases in 100 volumes is about 21 to 79. The air on the sea-shore and open heaths of Scotland is found to contain 21 volumes of oxygen in 100 of air, and on the top of the hills 20·98; in a suburb of Manchester it is 20·96 to 20·98; at Paris the mean quantity of oxygen for one year was 20·96; at St. John's, Antigua, it was

20·95; but in some hot climates it has been found as low as 20·3.

Besides these two principal ingredients, air also contains a small but perceptible proportion of *carbonic acid* gas, which is a chemical union of carbon with oxygen, and is $1\frac{1}{2}$ times as heavy as air. This gas is found in the air of the open country to be in the proportion of 3·36 volumes to 10,000 of air, while in towns where much coal smoke is produced, it varies from 4 to 6 or 7 volumes. Experiments show that when the quantity of this gas in 10,000 volumes of air exceeds 6 volumes, ill effects are produced on persons who breathe it; and as 15 cubic feet of carbonic acid are given off by every person in 24 hours, or nearly two-thirds of a cubic foot per hour, it is clear that the air in a room which is closely shut up must soon become poisonous to those who occupy it, and that it will require to be frequently changed. The burning of two sperm candles produces the same amount of this gas as one man does, or one candle yields ·3 cubic feet per hour. A gas burner consuming 3 feet of gas per hour produces 6 cubic feet of carbonic acid in the same time.

Another gas, which is found in minute quantities in air, is called *ozone*, the amount of which is very variable, more being found in country air than in the air of towns, and its proportion is greatest in spring and least in winter. The presence of ozone can generally be detected by delicate chemical tests on rainy days and during great atmospheric disturbances, but its maximum proportion is only one in 700,000 volumes of air. It is believed to be a condensed form of oxygen, possessing greatly increased chemical activity surpassing that of oxygen as much as oxygen surpasses air; 3 volumes of oxygen become condensed by electrical action into 2 of ozone, which can

again be expanded by heat to 3 volumes of oxygen. Atmospheric electricity is apparently a great generator of ozone. It has the power of destroying bad smells and the germs of contagion, by its rapid oxidization of organic matter.

82. Diffusion.—If a vessel is partly filled with water, and then a lighter liquid, such as oil, is poured on the top, it will be seen that the two liquids do not mix, but remain as perfectly distinct as if they were in separate vessels. When, however, we do the same with two gases of different density, they are found to *diffuse* through each other in a very short time, the heavier one rising into the lighter and the lighter one sinking into the heavier, until a uniform mixture is produced; the only difference being, that the velocity of diffusion of a heavy gas into a light one is less than that of a light gas into a heavy one, the rate of diffusion of gases into one another being inversely as the square roots of their relative densities. It is by diffusion that smells are rapidly conveyed in the air, and it is chiefly owing to this phenomenon that its composition remains nearly the same in all parts of the world.

If a tube closed at one end by a plug of plaster of Paris is filled with a gas, it will be found to pass out through the plaster (when quite dry) and mix with the surrounding air, and at the same time the air will pass into the tube through the plaster, so that the whole contents of the tube will soon be changed. In this way the external walls of a building assist in the ventilation of the rooms, it having been found by experiment that, with a difference of 4° Fahr. between the temperatures of the air inside and outside, from 5 to 6 cubic feet of air pass through a square yard of dry stone wall per hour, and about 8 feet through a brick wall; so that the larger the

external surface the better will be the ventilation. Wet walls are, however, impervious to the diffusion of air, and consequently we find newly-built houses to be less healthy than older ones, which have had time to dry. Mortar, when dry, is very porous to air. Diffusion through walls takes place more rapidly in winter than in summer.

83. VENTILATION.—The effect of heat on air and gases is to cause their expansion by $\frac{1}{491}$ of their bulk for each degree of the Fahrenheit scale. The amount of heat given off from the body and breath of an adult in a single hour is sufficient to produce a rise of 173° Fahr. in a chamber containing 150 cubic feet of air, provided that none of the heat is carried away by conduction. When the air at the bottom of a room is heated it becomes lighter in weight, and consequently ascends to the ceiling, parting with some of its heat to the colder air through which it passes. If there is an inlet at the bottom and an outlet at the top, a regular current will thus be produced by the cold air coming in below, to supply the place of the warm air which is ascending and passing out at the top. The object of ventilation of rooms is the removal of the superabundance of carbonic-acid, which has been generated therein by the persons occupying it and by the lights burning. Now, in order to prevent the proportion of this gas from exceeding 6 parts in 10,000, it is necessary that 3,000 cubic feet per hour of pure air should be supplied to each person in the room, so that if there is one person to every 500 cubic feet of space, it will be necessary to change the air of it six times per hour; 50 cubic feet per minute, or $\frac{5}{6}$ of a cubic foot per second, must then be passed into the room, so that, with an opening of $\frac{5}{6}$ of a square foot, or 120 square inches, the air must enter at a velocity of 1 foot per second, or ·682 mile per hour; with an inlet of half

this size, the velocity must be double; with one of 30 square inches, the velocity must be 4 feet per second, or 2·728 miles per hour; and if the inlet is 24 square inches, the velocity must be 5 feet per second, or 3·41 miles per hour, which is as great as can be endured with comfort by most persons without the sensation of a draught.

The pressure of the air on the surface of the earth being on an average about 15 lbs. per square inch, is equivalent to that of a column of air of uniform density 5 miles in height; consequently from the formula $v = 8 \sqrt{h}$, (77) we find that air would rush into a vacuum with a velocity *due* to the height of 5 miles, or

$$v = 8 \sqrt{26400} = 1300 \text{ ft. per second.}$$

The expansion of gases by heat being $\frac{1}{491}$ of their volume for every degree Fahr. that the temperature is raised, it follows that they have less density, or are lighter at high than at low temperatures, so that an upward current will set in when the air at the bottom of a room is warmed; and if there is an outlet at the top as well as an inlet of the same size at the bottom, the current will flow at a rate depending on the difference of temperature between the external and internal air, and on the height of the orifice of exit above the inlet; if h is this height, and a° the difference of temperature, then the expansion due to a° is $\frac{a}{491}$, and the height due to the velocity v, at which the cold air enters, is $h \times \frac{a}{491}$, so that we have,

$$v = 8 \sqrt{h \frac{a}{491}}$$

For example, if the height of exit is 25 feet above the

inlet, and the temperature of the air in the room 10° higher than that outside, we find

$$v = 8 \sqrt{\frac{25 \times 10}{491}} = 5{\cdot}7 \text{ feet per second.}$$

The quantity of air which enters will be v into the area of the opening in feet. Let the room be a cube of 25 feet, it is required to find the area of inlet in order that sufficient air may be admitted at the above velocity to change the air of the room six times in one hour. Here, the cubical contents being 15625 feet, we have to supply six times this quantity in the hour, or 26 cubic feet in every second, at a velocity of 5·7 feet; therefore, the inlet must have an area of $\frac{26}{5{\cdot}7}$, or, 4·56 square feet.

Where the air has to be admitted through pipes the velocity is diminished by the resistance of their surface, in the same way as in the case of water flowing through pipes (**78**), and the same coefficient of friction must be used. Bends and elbows also produce a reduction of velocity in the same proportion as we found in the case of water (**79**).

The value of h can be greatly increased without raising the height of the room, by connecting the outlet with a flue which may be carried up to any required height, and h will be the height of the top of this flue above the inlet; but, in order to prevent a down draught in the flue, which would disturb the arrangements for ventilation, it is sometimes necessary to heat the air at the bottom of the flue by means of a furnace or gas burners, so as to cause a constantly ascending current in the flue, and we must then take $a°$ as the difference between the temperature in the flue and that of the entering air. An increase

of velocity may also be produced by means of a revolving cowl or fan, which creates a partial vacuum in the flue.

It is a mistake to suppose that because carbonic acid gas is much denser than air it will necessarily be found in greater quantities near the floor of a room than at the top, for the law of the *diffusion* of gases (**82**) shows us that it mixes rapidly with the rest of the air, which becomes nearly uniform in composition; in fact, it is often found in crowded theatres that the proportion of carbonic acid is greater in the upper than the lower strata of the air, owing to the high temperature at which it is exhaled from the lungs of the audience.

84. Pressure of Wind.—In discussing the question of strains upon roofs (**50**) the pressure arising from the action of wind on their exposed surface was taken at nearly double that of the whole structure itself, so that this becomes a very important element in the erection of buildings, and one that is too often ignored, with the natural result of the destruction of lofty chimney-stacks, towers, spires, and roofs, by heavy gales of wind. In exposed situations, and at a considerable elevation from the ground, the velocity of the wind is much greater than where there are obstacles to its progress, such as trees, and the friction offered by the surface of the ground.

When the wind blows against a plane surface at right angles to its direction, the pressure it produces is proportional to the area of the plane and to the square of its velocity; so that, if we can determine accurately the pressure for any known velocity, we are able to find it for any other velocity. It was ascertained by Smeaton that a current of air moving with a velocity of 35 miles an hour, or 51 feet per second, produced a pressure of 6 lbs. per square foot on a plane at right angles to its direction,

so that, when the velocity is 70 miles an hour, or 102 feet per second, the pressure by the above law will be 24 lbs. per square foot. If we put p for the pressure per square foot on a plane at right angles to the direction of the wind moving with a velocity v, we have

$$\frac{p}{24} = \left(\frac{v}{70}\right)^2, \text{ or } p = \frac{v^2}{204}$$

we can thus obtain the pressure p for any velocity v, as shown in the following Table.

v.		p.
Miles per Hour.	Feet per Second.	Lbs. per Foot.
20	29	1·96
30	44	4·41
40	58	7·84
50	73	12·25
60	88	17·64
70	103	24·00
80	117	31·36
90	132	40·00

When the wind strikes on a plane inclined at an angle to its direction, the pressure per square foot will be less according to the angle of inclination; and by the principle of the composition of velocities, the velocity perpendicular to a plane making an angle a with its direction, is $v \,.\, \sin a$, which we must substitute for v in the above expression for p; or if p' is the pressure on the plane at the angle a, p that perpendicular to its direction, we have

$$p' = \frac{v^2 \,.\, \sin^2 a}{204} = p \,.\, \sin^2 a$$

Dr. Hutton obtained an empirical formula for the pressure on inclined surfaces from experiments with a

small plane made to move at different angles of inclination, but the results which it gives appear to be at variance with the principles of mechanics, since it makes the pressure on a plane inclined at 57° the same as on one that is vertical, the discrepancy being probably due to the creation of a partial vacuum behind the moving plane, which would not occur with such a large surface as a roof exposed to the pressure of the wind.

The pressure on a foot of surface of a plane inclined to the direction of the wind's motion at any angle a can be determined by multiplying the value of p found in the above Table by the value of $\sin^2 a$ given in the following one. The highest pitch of roof is about 60°, and, with the wind moving horizontally at 80 miles an hour, the pressure on a square foot would be $\cdot 75 \times 31\cdot 36$, or $23\frac{1}{2}$ lbs., which may be considered as the greatest ever likely to be put upon a roof in this country.

$a°$	$\text{Sin.}^2 a$
60°	·750
$52\frac{1}{2}$°	·630
45°	·500
$37\frac{1}{2}$°	·370
30°	·125
$22\frac{1}{2}$°	·146
15°	·067

In calculating the strains, however, produced by wind upon a roof, it must be remembered that the pressure is not uniform over the whole roof, but only upon one side at a time, so that a racking strain is put upon it, which exerts a far greater effect on the framework than the mere addition of the same amount of pressure all over the roof. The mode of drawing the diagram of stress for the wind on

one side of a roof is shown with an example in the present author's treatise on "Carpentry" (Weale'sSeries), page 110, and also in the Appendix to his "Practical Geometry," 2nd. edit.; but it is evident that when the angle a does not exceed 30° the effect of the wind when blowing horizontally upon the roof is but slight, the actual pressure being, by the Table, only one-sixth of the pressure on a roof of 60°, or, $p' = 4$ lbs. The current of wind, however, does not always move horizontally, but is often in high gales directed obliquely downwards, in which case it might produce a greatly increased pressure upon a roof of low pitch.

In Chapter I. (7) we have shown how the effect of wind on a wall or chimney can be calculated; in the same manner we can determine the thickness to be given to a wall which stands alone, such as a chimney-stack rising high above a roof, in order that a pressure say of 40 lbs. per foot shall not overturn it. Let h be its height, t its thickness, δ the weight of a cubic foot; then the weight of 1 foot length of the wall is $\delta . h . t$, and the pressure on the surface is $40\,h$. Equating the moments of these forces about the bottom edge of the wall, we have

$$\delta . h \frac{t^2}{2} = 40 \frac{h^2}{2}$$

$$\text{or, } t = \sqrt{\frac{40\,h}{\delta}}$$

when the wall is just about to turn over; so that, for stability, t must be greater than this equation gives.

For example, let $h = 27$ feet, $\delta = 120$ lbs., then we have

$$t = \sqrt{\frac{40 \times 27}{120}} = 3 \text{ feet.}$$

The pressure of the wind upon a circular tower is less than on a square one of the same diameter, as we may suppose its surface to consist of a number of small planes whose angles of inclination with the direction of the wind increase from the middle towards the outside, so that the pressure decreases according to the law above stated. By means of the Integral Calculus we find that the pressure on the surface of a circular cylinder is two-thirds of that on a square prism of same diameter.

CHAPTER VIII.

LIGHTNING-CONDUCTORS.

85. Lightning-Conductors are rods of metal attached to the highest parts of a building to convey the electric fluid (as it is popularly called) from the clouds to the earth, and thereby prevent it from striking and damaging the structure. The identity of the lightning-flash with the electricity of the Leyden jar was established by Dr. Franklin and other philosophers about the middle of the last century, and we are thus enabled by experiments in the laboratory to determine the best means of protecting buildings from the destructive effects of lightning.

If we rub a glass rod with a piece of dry silk and then hold it near a pith ball suspended by a silken thread, the ball is at first attracted, but, if allowed to touch the glass, it is immediately repelled. If we now rub a stick of sealing-wax with a piece of dry flannel and bring it near the pith ball just repelled by the glass, it will be attracted to the sealing-wax. In this experiment the glass is said to be charged with positive and the sealing-wax with negative electricity; and it is found that bodies charged with the same kind of electricity, whether positive or negative, repel one another, while those charged with opposite kinds attract each other. It is also found that only that part of the rod which was rubbed is electrified, the opposite end being quite free from electricity, and

that those substances which produce it when rubbed, do not conduct it, or at least in a very imperfect degree. On the other hand the metals, unless "insulated," will not produce electricity when rubbed, but, if it is communicated to one end of a metallic rod, it is immediately conveyed to the other end. Hence it is that metals have been called "non-electrics," or "conductors," while glass, silk, flannel, and sealing-wax, have been termed "electrics," or "non-conductors." No substances, however, are perfect conductors or non-conductors, as all offer more or less resistance to the passage of electricity, and even glass, which is one of the worst conductors at an ordinary temperature, is a moderately good one when heated to redness. Water is a conductor at ordinary temperatures, but, when converted into ice, it conducts very badly. All metals are good conductors, although some much better than others, copper and silver being the best, while zinc offers three times as much resistance as copper does, iron seven times as much, and lead twelve times as much. When a conducting body is supported on a non-conducting substance it is said to be "insulated," so that the electricity it contains cannot be carried off to the earth.

If we take an insulated brass cylinder with spherical ends and bring it near the prime conductor of an electrical machine charged with electricity, we find that its natural electricity is at once decomposed so that the end next the charged conductor becomes charged with the opposite kind of electricity, while the further end is charged with the same kind as that in the conductor. The action which here takes place is called "Induction," and, wherever one kind of electricity is produced in any body by any means, the opposite kind will be found to be *induced* in neighbouring bodies. Thus, when a glass rod

is rubbed with silk, the glass becomes charged with positive and the silk with negative electricity in equal quantities.

If now we connect the brass cylinder with the earth by a metallic chain, and move it near the charged conductor, a flash will pass from the positively electrified body to the one negatively electrified, the length of the spark depending partly on the quantity of electricity which one contains more than the other, or, as it is termed by electricians, on the "difference of potential," and partly upon the density of the intervening fluid or "dielectric," rarified air offering a much less resistance to its passage than air of a denser kind. If, however, the cylinder presented to the conductor is terminated by a sharp metallic point, no sudden flash will pass, but the electricity of the cylinder will be concentrated on the point at a high degree of tension, and will fly off to the conductor until an equilibrium is established between the two kinds of electricity in a gradual manner by what is called the "brush discharge."

It is upon the phenomena of Induction and brush discharge above described that the principle of the lightning-conductor depends. A thunder-cloud may be regarded as an enormous electrical conductor usually charged with positive electricity, which as it approaches any object on the earth decomposes the electricity it contains, inducing negative electricity to collect upon it; and as soon as the "potential" of the cloud is sufficient to overcome the resistance of the air a discharge will take place between the cloud and the nearest object, and if that happens to offer great resistance to its passage, or is a bad conductor, it will be shattered by the shock. If, on the other hand, it is a good conductor, or is provided with a continuous

rod of a good conducting substance, the charge will be conveyed in safety to the earth, and the equilibrium between the two kinds of electricity will be restored. By providing this conductor with a point or points the lightning is prevented from striking, and a brush discharge of negative electricity takes place which relieves the cloud of its superabundance of positive electricity, and restores the equilibrium which was disturbed. In comparing however the action of points on a lightning-conductor with their effect in the laboratory with an ordinary electrical machine, it must be remembered that the thunder-cloud is infinitely larger, and contains an infinitely greater amount of electricity than the charged conductor of the most powerful machine, so that a ball even as large as that on the top of St. Paul's would be but a point in comparison to the size of the cloud. It seems therefore quite unnecessary to make the lightning-conductor terminate with a sharp point in order to obtain a brush discharge, where the amount of electricity we have to deal with is so enormous.

The best mode of protecting buildings from being struck is to connect together all the external metal work throughout, and also with the ground, taking especial care that the highest parts are connected with the rest by good metallic conductors. For example, a church having a lead-covered spire terminated by a metal vane, and also having its roof covered with lead, and communicating with the ground by means of metal rain-water pipes, will be effectually protected if the lead on the spire is connected by a metallic rod with that on the roof. In the case of stone towers, spires, or chimney-stacks, a stout rod of metal must be attached having one end carried up a few feet above the top of the masonry, and the lower

end buried in the ground to a sufficient depth to insure its being always in a moist soil, or else into a well of water if one is near at hand. It is advisable to surround the end in the ground with wood ashes or coke, which act as conductors, and also serve to protect the metal. The action of the conductor is to neutralise the positive electricity of the air and clouds, to prevent the accumulation of negative electricity on the earth's surface, as well as to restore the clouds to their condition of electrical equilibrium, and thus prevent the danger arising from the discharge of flashes of lightning.

86. As regards the material of which a lightning-conductor should be formed, it is evident that the metal which offers the least resistance to the passage of electricity is the best for this purpose, and, as it is usually placed on the outside of a building, and is much exposed to the weather, the metal should be one not liable to corrosion. Rods of iron have been used, but the rapid decay of this metal renders it an undesirable material, as, in case of any part of the rod being broken, or reduced in sectional area, much damage might be occasioned to the building if struck by lightning. As copper offers less resistance than iron, and is not so liable to corrode, this metal is now universally employed in place of iron for lightning-conductors.

The sectional area of the conductor must depend, to some extent, on the height of the building on which it is placed, as the resistance is directly as the length of the conductor, and inversely as its area of section. It has been found by experience that a copper rod $\frac{3}{4}$ inch square will effectually protect the loftiest building, so that for those of ordinary height a smaller one will suffice, but should never be less than $\frac{1}{2}$ inch diameter. Sir W. Snow

Harris, who paid great attention to the subject, recommended that the rod should be of copper tubing from 1 inch to 2 inches diameter, and $\frac{1}{5}$ inch thick, put together in lengths of 10 feet by being screwed into intermediate pieces of thick tube 2 inches long. There is, however, a difficulty in adapting such a rod to the irregularities in the outline of most buildings, and since the invention of rope made of copper wires twisted together, this material has been generally employed both from its being continuous throughout, and being easily bent over any projections, so that it can be carried over any part of the building much more easily than a solid rod or hollow tube.

The conductor should not merely be carried straight down from the highest point of a building to the ground, but should also be connected with all the metal-work of the roofs, otherwise there is danger of lateral discharges which may inflict serious damage. The rod should be placed in close contact with the wall, and all attempts at insulation by means of glass or porcelain sockets should be avoided, as they are both useless and disadvantageous; for the electricity will naturally choose the path that offers least resistance, and, even if anything occurred to cause it to leave the rod, it is evident that a flash of lightning which will pass through several hundreds of yards of air will not be affected by a few inches of an insulator.

The extent of building which a single conductor will defend is a matter on which there are diversities of opinion. Some French authorities have asserted that a rod whose height is h, will protect a circular area whose radius is $2h$; so that a rod 100 feet high should protect a space around it of 200 feet radius. Such a rule probably holds good when lightning is discharged vertically from a

cloud, but, as it is often discharged in an oblique direction, it may strike a lower portion of a building at a distance from the conductor. For instance, a tree was struck although only 52 feet from a conductor erected upon a house, while the house was untouched; in another case a house was struck at a point only 40 feet from a conductor. Several other cases have occurred which seem to show that very little reliance can be placed on this rule, and that where we have large ranges of straggling buildings it is necessary to distribute connected conductors through them, and to unite these with pointed rods placed at the most prominent parts of the buildings.

Much information as to the value and importance of lightning-conductors will be found in a treatise published in 1843 by Sir W. Snow Harris on the means of protecting buildings from the destructive effects of lightning.

APPENDIX.

TABLE I.

Specific Gravity and Weight of Building Materials.

The following is a Table of the specific gravity and weight per cubic foot of some of the principal materials used for building purposes :—

Materials.	Spec. grav. & weight in ozs. of 1 cub. ft.	Weight of 1 cub. ft. in lbs. av.
Water	1000	62·50
Beech, dry	690	43·12
Elm, seasoned	553	34·56
Fir (Norway spruce)	512	32·00
,, (white American spruce) . . .	465	29·06
,, (Memel), dry	544	34·00
,, (Riga), dry	466	29·12
Pine (American), dry	368	23·00
,, (American pitch), dry	936	58·50
,, (,,), seasoned	741	46·31
Mahogany (Spanish), dry	852	53·30
,, (Honduras), dry	560	35·00
Oak (Old English), dry	625	39·06
,, (English), seasoned	777	48·56
,, (Riga), dry	688	43·00
,, (Dantzic), seasoned	755	47·24
,, (American), red	752	47·00
,, (,,), white	840	52·50
Teak, dry	745	46·56
,, seasoned	657	41·06
Granite (Guernsey)	2999	187·47
,, (Aberdeen)	2664	166·50
,, (Cornish)	2624	164·00
,, (Dublin)	2713	169·57
Slate (Welsh)	2888	180·50

Materials.	Spec. grav. & weight in ozs. of 1 cub. ft.	Weight of 1 cub. ft. in lbs. av.
Slate (Westmoreland)	2768	173·00
,, (Cornwall)	2512	157·00
SANDSTONES :—		
Abercarne (Monmouth)	2688	167·93
Pyotdykes (Dundee)	2601	162·57
Glammis (Forfar)	2578	161·20
Munlochy (Ross)	2569	160·60
Mylnefield (Dundee)	2561	160·07
Auchray (ditto)	2542	158·90
Lochee (ditto)	2538	158·70
Scotgate-head (Huddersfield)	2528	158·00
Colford (Gloster)	2491	155·72
Longwood (Huddersfield)	2455	153·44
Elland (Halifax)	2452	153·28
Park-spring (Leeds)	2418	151·11
Talacre (Flint)	2403	150·27
Mansfield, red	2378	148·66
,, white	2344	146·51
Darley-dale, Derby	2372	148·25
New Leeds (Yorkshire)	2360	147·50
Chepstow (Monmouth)	2348	146·78
Bewdley (Salop)	2336	146·00
Kenton (Northumberland)	2320	145·00
Giffneuk (Glasgow)	2303	143·90
Hookstone (Harrogate, Yorkshire)	2282	142·62
Stainton (Durham)	2280	142·53
Bramley-fall (Leeds)	2276	142·23
Catcraig (Linlithgow)	2267	141·70
Cragleith (Edinboro')	2266	141·63
Humbie (,,)	2244	140·23
Meanwood (Leeds)	2238	139·88
Redgate (Durham)	2234	139·62
Hunger-hill (Belper)	2175	135·95
Gatherley (Yorkshire)	2173	135·82
Pensher (Durham)	2148	134·35
Hollington (Stafford)	2129	133·10
Duffield (Derby)	2127	132·90
Longannet (Perth)	2108	131·73
Heddon (Northumberland)	2092	130·73
Morley-moor (Derby)	2089	130·54
Whitby (Yorkshire)	2027	126·70
Tixall (Stafford)	1993	124·56
Calverley (Kent)	1889	118·06
Tisbury (Wilts)	1778	111·13
Gatton (Reigate)	1650	103·10

Materials.	Spec. grav. & weight in ozs. of 1 cub. ft.	Weight of 1 cub. ft. in lbs. av.
MAGNESIAN LIMESTONES :—		
Bolsover (Derby)	2427	151·69
Anston (Yorkshire)	2304	144·00
Roche-abbey (Bawtry, Yorkshire)	2228	139·20
Huddlestone (Sherburne ,,)	2205	137·83
Park-nook (Doncaster ,,)	2198	137·20
Brodsworth (,, ,,)	2138	133·66
Cadeby (,, ,,)	2026	126·60
OOLITES :—		
Wass (Thirsk, Yorkshire), hard	2600	162·50
,, (,, ,,), soft	2267	141·68
Aubigny (France)	2400	150·00
Ancaster (Lincoln)	2229	139·30
Barnack (Stamford, Lincoln)	2188	136·78
Taynton (Oxon)	2175	135·93
Windrush (Gloster), hard	2175	135·93
,, (,,), soft	1891	118·18
Portland (Waycroft)	2169	135·56
,, (Roach)	2028	126·80
Doulting (Wilts)	2147	134·25
Haydor (Lincoln)	2135	133·48
Ketton (Rutland)	2045	127·81
Caen (France), average	2000	125·00
Box (Bath)	1968	123·00
Coombe-down (Bath)	1856	116·00
LIMESTONES :—		
Kentish-rag (Maidstone)	2666	166·60
Hopton-wood (Derby)	2535	158·45
Chilmark (Wilts), hard	2518	157·38
,, (,,), soft	2426	151·60
Blue-lias	2467	154·18
Seacombe (Purbeck)	2416	151·03
Chalk	2315	144·68
Hamhill (Somerset)	2268	141·75
Hildenley (Malton, Yorkshire)	2202	137·65
Barnack (Lincoln)	2188	136·76
Sutton (Glamorgan)	2176	136·00
Beer (Devon)	2108	131·75
Totternhoe (Beds)	1864	116·50
Earth, loamy	2016	126·00
,, common	1984	124·00
,, clay	1919	119·93
,, gravel	1749	109·32
,, loose	1520	95·00

Materials.	Spec. grav. & weight in ozs. of 1 cub. ft.	Weight of 1 cub. ft. in lbs. av.
Concrete	2240	140·00
Brickwork	1792	112·00
Mortar	1744	109·00
Iron, wrought	7680	480·00
,, cast	7120	444·00
Steel	7840	490·00
Lead, milled	11,407	712·93
Copper, sheet	8785	549·06
Brass, cast	8100	506·25
Bell-metal	8760	547·50
Zinc	7028	439·25
Glass, plate	2760	172·50
,, crown	2520	157·50
Brick, stock	1841	115·06
,, red	2168	135·50
Tile, plain	1815	113·43

TABLE II.

Powers of Numbers.

a	$a^{1·63}$	a^2	a^3	$a^{3·5}$	$a^{3·55}$	a^4
1	1·0	1·0	1·0	1·0	1·0	1·0
1·25	1·4	1·6	1·95	2·2	2·2	2·4
1·5	1·9	2·3	3·38	4·1	4·2	5·1
1·75	2·5	3·1	5·4	7·1	7·3	9·4
2	3·1	4	8	11·3	11·7	16
2·25	3·8	5·1	11·4	17·1	17·8	25·6
2·5	4·5	6·3	15·6	24·7	25·8	39·1
2·75	5·2	7·6	20·8	34·5	36·3	57·2
3	6·0	9	27	46·8	49·4	81·0
3·25	6·8	10·6	34·3	60·5	65·6	111·6
3·5	7·7	12·3	42·9	80·2	85·4	150·1
3·75	8·6	14·1	52·7	102·1	109·1	197·8
4	9·6	16	64	128·0	137·2	256·0
4·25	10·6	18·1	77	158·3	170·1	326·3
4·5	11·6	20·3	91	193·3	213·3	410·1
4·75	12·7	22·6	105	233·6	252·5	509·1
5	13·8	25	125	279·0	303·6	625·0
5·25	14·9	27·6	145	331·6	360·2	759·7
5·5	16·1	30·3	166	390·2	424·9	915

a	$a^{1\cdot 63}$	a^2	a^3	$a^{3\cdot 5}$	$a^{3\cdot 55}$	a^4
5·75	17·3	33·1	190	455·9	497·5	1093
6	18·6	36	216	529·0	578·8	1296
6·25	19·8	39·1	244	610·4	668·9	1526
6·5	21·1	42·3	275	700·2	768·9	1785
6·75	22·5	45·6	308	799·0	879·1	2076
7	23·9	49	343	907·5	1000	2401
7·25	25·3	52·6	381	1026·1	1133	2763
7·5	26·7	56·3	422	1155·3	1278	3164
7·75	28·2	60·1	465	1295·8	1436	3607
8	29·7	64	512	1448·2	1607	4096
8·25	31·2	68·1	562	1612·8	1792	4632
8·5	32·7	72·3	614	1790·5	1993	5220
8·75	34·3	76·6	670	1981·7	2209	5862
9	35·9	81	729	2237·9	2441	6561
9·25	37·6	85·6	791	2407	2690	7321
9·5	39·2	90·3	857	2643	2957	7959
9·75	40·9	95·1	927	2894	3243	9037
10	42·7	100	1000	3162	3548	10000
10·5	46·2	110·3	1158	3751	4219	12155
11	49·8	121	1331	4414	4977	14641
11·5	53·6	133	1521	5158	5828	17490
12	57·4	144	1728	5986	6778	20736
12·5	61·4	156	1953	6905	7835	24415
13	65·4	169	2197	7921	9005	28561
13·5	69·6	183	2460	9040	10296	33214
14	73·8	196	2744	10267	11716	38416
14·5	78·2	210	3049	11610	13270	44206
15	82·6	225	3355	13071	14967	50325
15·5	81·2	241	3724	14661	16814	57719
16	91·8	256	4096	16384	18820	65536
16·5	96·5	272	4492	18243	20992	74116
17	101·3	284	4913	20257	23340	83521
17·5	106·2	307	5360	22420	25870	93790
18	111·2	324	5832	24743	28590	104976
18·5	116·3	342	6332	27233	31510	117133
19	121·4	361	6859	29897	34640	130321
19·5	126·7	381	7415	32742	37985	144583
20	132·0	400	8000	35778	41558	160000
20·5	137·5	420	8615	39006	45365	176608
21	143·0	441	9261	42439	49418	194481
21·5	148·5	462	9939	46083	53723	213689
22	154·2	484	10648	49942	58290	234256
22·5	160·0	506	11391	54030	63130	256298
23	165·8	529	12167	58353	68257	279841
23·5	171·7	552	12978	62914	73672	304983
24	177·7	576	13724	67723	79387	329376
24·5	183·8	600	14707	72795	85420	360322
25	190·0	625	15625	78125	91768	390625
25·5	196·2	650	16585	83732	98450	422918

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